PROGRESSIVE COLLAPSE: COMPUTER ANALYSIS OF BEAMS AND DETAILING

HONORS THESIS

Presented in Partial Fulfillment of the Requirements to Graduate with Distinction from the Civil and Environmental Engineering and Geodetic Science Department at The Ohio State University

By

Daniel Justin Morone

The Ohio State University

2010

Undergraduate Honors Examination Committee:

Dr. Halil Sezen

Dr. Shive Chaturvedi

Approved by:

____

____

Advisor

Civil Engineering Honors Undergraduate Program

ABSTRACT

The term 'progressive collapse' can be simply defined as the ultimate failure or proportionately large failure of a portion of a structure due to the spread of a local failure from element to element throughout the structure. Progressive collapse can be triggered by manmade, natural, intentional, or unintentional causes. Fires, explosions, earthquakes, or anything else causing large amounts of stress and the failure of a structure's support elements can lead to a progressive collapse failure. Progressive collapse is a complicated dynamic process where the collapsing system redistributes the loads in order to prevent the loss of critical structural members. For this reason beams, columns, and frame connections must be designed in a way to handle the potential redistribution of large loads. Some of the more famous examples of progressive collapse phenomena include the collapse of the World Trade Center towers due to terrorist attack, the bombing of the Murrah Federal Building in Oklahoma City, and the collapse of the Ronan Point building due to a gas explosion. Through research being done, such as that in this study, progressive collapse can be better prepared for and possibly prevented in the future.

This project involves use of two main computer programs to perform analysis on a reinforced concrete structure. MATLAB and SAP2000 are used to determine the total amount of steel rebar required to prevent progressive collapse at midspan of a continuous beam section where a column loss has occurred. The research results provide insight into the minimum amount of reinforcing steel actually needed to achieve a demand to capacity ratio of approximately 1.0 and prevent collapse in the event of a single column loss. Several relationships are developed between span lengths, distributed loading, column loading, and steel required. The maximum allowable loads are calculated to show how to best prevent progressive collapse. The results are obtained using different models where combinations of pinned end and fixed end support configurations are used along with analysis of nominal moment strength of the beam versus the plastic moment strength of the beam. Ultimately, tables and graphs are developed that could eventually be used in design codes where there are currently very limited or no specific rules or guidelines directed towards prevention of progressive collapse.

Acknowledgements

First I would like to thank my parents, Dan Morone and Lisa Morone, for the constant support and guidance they have given me all of my life. Without them I may never have had the personal drive to do everything to the best of my abilities and reach for my highest goals. They have had a huge effect on the making me the person I am proud to be today.

Next I want to thank all of my family and friends who have also been there to support me in the toughest of times. It is difficult to imagine where I would be without all that they have given me. From academia to personal matters, I have learned a lot from this group and will never forget how they have helped me get where I am.

I want to thank the faculty members at The Ohio State University who have assisted me in many ways. I am grateful to all of the professors who have aided me in learning as much as possible on a wide variety of topics at this college. I also want to show my great appreciation to those who make the Land Grant Opportunity Scholarship possible. They have certainly created the possibility of attending college a reality for me and not a day passes where I do not think of how grateful I am for their financial support.

Finally I would like to thank my advisor, Dr. Halil Sezen, for guiding me through the process of conducting research and sharing his wisdom of structural engineering to make better engineers of all of us in his classes. He has truly played a major role in my education and has had a great effect on my college career.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

The term 'progressive collapse' can be simply defined as the ultimate failure or proportionately large failure of a portion of a structure due to the spread of a local failure from element to element throughout the structure. Progressive collapse can be triggered by manmade, natural, intentional, or unintentional causes. Fires, explosions, earthquakes, or anything else causing large amounts of stress and the failure of a structure's support elements can lead to a progressive collapse failure. Progressive collapse is a complicated dynamic process where the collapsing system redistributes the loads in order to prevent the loss of critical structural members. For this reason beams, columns, and frame connections must be designed in a way to handle the potential redistribution of large loads. Some of the more famous examples of progressive collapse phenomena include the collapse of the World Trade Center towers due to terrorist attack (FEMA, 2002), the bombing of the Murrah Federal Building in Oklahoma City (FEMA, 1996), and the collapse of the Ronan Point building due to a gas explosion (Pearson, 2005). Through research being done, such as that in this study, progressive collapse failures can be better prepared for and possibly prevented in the future.

1.2 Research Significance

This research is important because while progressive collapse is not a common event, its effects can be catastrophic if design methods and specifications are not good enough to slow or stop the spread of damage throughout the structure. Advancements in research on this subject can help prevent large amounts of damage from spreading after the initial triggering event has occurred. In addition to having better damage control, buildings designed against progressive collapse provide more safety to those who use and occupy the structure. Many lives can be saved if damage is contained to the area of initial structural element loss rather than allowing the damage and its effects to spread through the building and collapse large areas. With the possibility of attacks or terrorists acts at its current levels, this study can provide significant information that may be used in future design methods to prevent progressive collapse in various situations.

1.3 Objectives and Scope

This project uses two main computer programs to perform analysis on a reinforced concrete structure. MATLAB (2009) and SAP2000 (2010) are powerful computational programs that are used to determine the total amount of steel rebar required to prevent progressive collapse at the midspan of a continuous beam section where column loss has occurred. The minimum amount of reinforcing steel actually needed is calculated to achieve a demand to capacity ratio of approximately 1.0 and to prevent collapse in the event of a single column loss. Several relationships are developed between span lengths, distributed loading, column loading, and steel required. The maximum allowable loads are calculated to show how to best prevent progressive collapse. The results are obtained using different models where combinations of pinned end and fixed end supports are used with nominal moment strength of the beam and the plastic moment strength of the beam are used and compared. Ultimately, tables and graphs are developed that could be used in design codes where there are currently limited rules or guidelines directed towards prevention of progressive collapse.

1.4 Organization

The research study begins with Chapter 1 which includes an introduction to the project. The problem statement, significance of research, and objectives and scope of the project are described. Chapter 2 includes a literature review of studies performed by other researchers and engineers on the topics related to progressive collapse of reinforced concrete structures. Chapter 3 gives a more detailed explanation of the problem statement and provides more details about the specific purpose of the research performed. Chapter 4 gives a description of the properties of the materials used for analytical purposes. Also the details of the computer models used are described in this chapter. Chapter 5 discusses how the models were used and all of the assumptions used to create and run the computer programs. Chapter 6 presents the analytical results found by running the computer models. Chapter 7 gives an interpretation of the results and describes how they may be used in practice for certain applications. Chapter 8 provides the summary and conclusions of this project. Finally, several appendices are included to provide a comprehensive set of all data and results obtained from the computer programs.

CHAPTER 2

LITERATURE REVIEW

Giriunas (2009) did a study involving the comparison of real building behavior to that of a computer model he developed on the computer program SAP2000. Giriunas placed strain gauges throughout various places in the structure to gather physical data of the building's response to the loss of a sequential set of columns. While his experiment dealt with a steel framed structure, the information provided by his study gives great insight into the steps used to gather experimental data and how to use it to determine the credibility and accuracy of a specific analysis method.

Hayes et al. (2005) developed some ideas of strengthening for earthquake conditions and applied it to an analysis case for the building in the Oklahoma City bombing event. Three different strengthening schemes were used and the detailing was updated to current codes of the time. It was determined that strengthening the perimeter elements increased progressive collapse resistance while strengthening internal elements was much less effective at this task. This study gives information in the connection of earthquake resistance to strengthening for progressive collapse as well as using computer analysis to compare several different setups of the building.

Zhongxian and Yanchao (2008) produced a study comparing the direct simulation method to the alternative load path method. It was determined that the direct simulation method was much more accurate but required a great deal of time and knowledge about structural dynamics while the alternate path method was very simple but much more inaccurate. A new method was proposed to give accuracy close to the direct simulation method but takes much less time while still including blast damage effects. This study is important because it shows the inaccuracies of the alternate path method which is used in this project and provides alternatives to potentially increase the accuracy of the results.

In a study by Sasani and Kropelnicki (2008) a 3/8 model of a building was produced and tested and compared with a detailed finite element model of the structure. Many different details were analyzed to determine the adequacy of the structure. The finite element model (FEM) was compared to a demand capacity ratio (DCR) method and determined that the DCR method is overly conservative. This study proves to be useful because the DCR method is used in this project and having knowledge of it being conservative may help explain certain results obtained.

Talaat and Mosalam (2009) performed a study that provides an algorithm for finite element codes. Probability curves for ultimate and partial collapse were developed empirically. Many issues outside of the scope of this project were discussed but usage of the ideas behind the probability curves for collapse could prove useful.

Bao et al. (2008) performed a study using a macromodel-based approach to simulate progressive collapse. A specific focus of this study was the behavior of joints in the frames. Also analyzed are different frames of which some have seismic strengthening. It was found that frames designed for high seismic areas were less susceptible to progressive collapse. Once again this is a useful study for modeling and comparing seismic detailing effects against progressive collapse.

A study by Tsai and Lin (2008) follows a linear static model that is recommended by the General Service Administration (GSA, 2003) to determine the adequacy of an earthquake-resistant building. It is shown how this simple analysis can be performed and suggested that the GSA may need to include inelastic dynamic effects in its guidelines. A curve for nonlinear static analysis cases were developed and could predict the resistance of a building. This study may be useful because its relative aim is similar to that of this project. Static cases and predicting resistance of a building are within the scope of this project.

Pekau and Cui (2006) performed a study using a distinct element method program to model the effects of precast panel shear walls with respect to progressive collapse. It was found that if seismic requirements were met by the shear walls, then it would automatically meet the demands needed for progressive collapse. This study could prove to be very important by its suggestion that using these shear walls and meeting seismic codes could lead to the prevention of many cases of progressive collapse.

Abruzzo et al. (2006) performed a study describing the assessment of an existing building. It shows through usage of the alternate path method that while the building meets the ACI 318 (2008) integrity requirements, it is still significantly vulnerable to progressive collapse failure. This study is also closely related to this project and should be useful in analyzing real buildings.

In a study by Mohamed (2009) the dimensions of damage limits from the Department of Defense (DOD) are analyzed using the alternate path method. This study focused on the importance of 3-D analysis for torsional effects and different types of frames. This may be useful in this project in terms of using 3-D modeling for analysis procedures.

CHAPTER 3

DISCUSSION OF PROBLEM STATEMENT AND SAP2000 MODEL

3.1 Progressive Collapse of Building Frames

The purpose of this project is to determine reinforcement detailing information and the amount of reinforcing steel required in the beam that would be required to prevent progressive collapse of the structure in the event that one column is lost under a certain span of the beam. Figure 3.1 below shows the general setup for the model being develop to analyze.

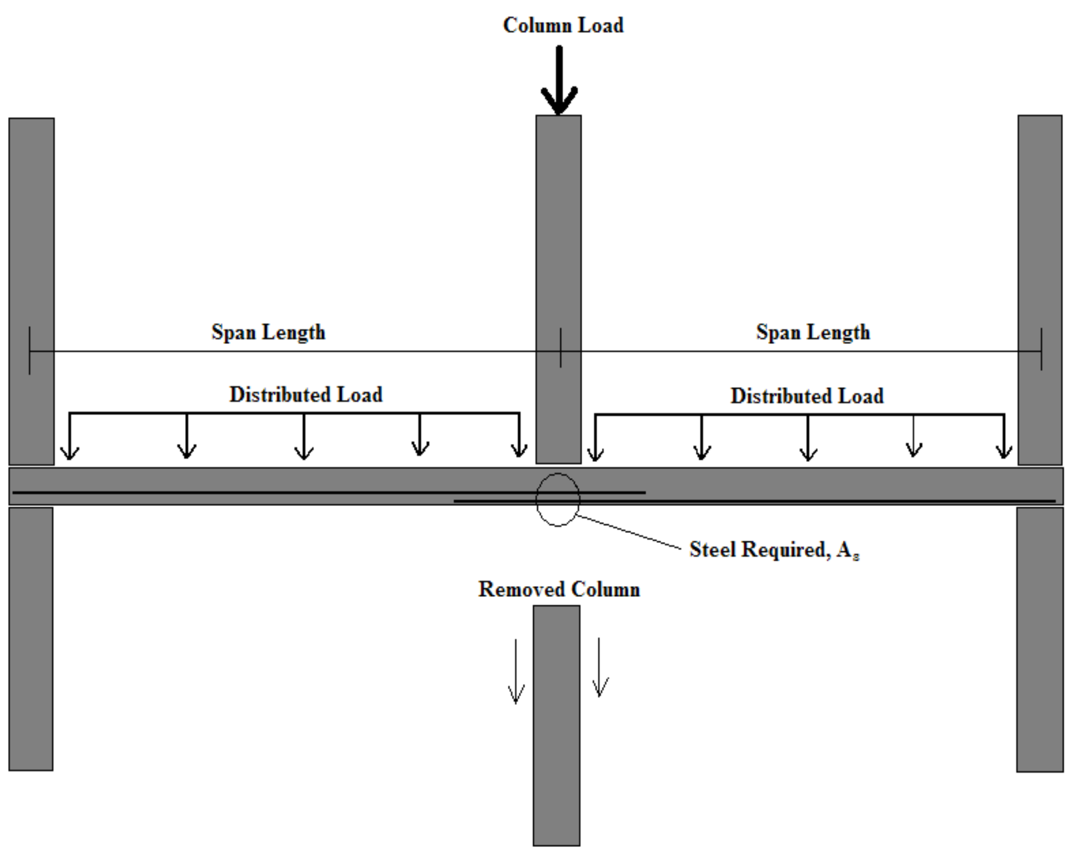

Figure 3.1: General model setup for computer analysis

For the purpose of this project, it is assumed that progressive collapse occurs if the beam supporting the large span above the lost column fails. This is assumed for two reasons. First of all, if the beam stretching the span between the remaining columns were to fail, a very large area of damage would result as it is unlikely the slab could support all of the loading especially if it was able to cause the beam to fail. The second reason this would qualify as progressive collapse is that once the beam failed, the slab would likely fail and then all of the loads would have to be redistributed throughout the structure and would have a good change of causing failure in other members of the frame. This would be especially true if a column above the ground floor were to be removed from the structure since the loads could drop down onto the next beam and likely cause it to fail also. With progressive collapse now defined with more detail for the purpose of this project, the manner in which analysis is to be conducted can be explained in more detail.

Since obtaining an actual building for test purposes was not a possibility for this project, a computer model of a building was created. This building was represented in SAP2000 as a two-dimensional frame with three stories in height and three bays in width. The frame, shown in Figure 3.2, was created in order to perform analysis on a general reinforced concrete structure with various loading conditions applied to its members. This study focuses on gravity loads being applied to the frame as distributed loads acting directly on the continuous beams. Seismic loads are not considered as the probability of an earthquake occurring at the same time as an unrelated column failure is extremely low.

3.2 SAP2000 Frame Models and Analysis

As shown below in Figure 3.2 and Figure 3.3, two frame models are created in the SAP2000 analysis program. Figure 3.2 shows the original setup of the frame representing a typical structure with all members intact. Figure 3.3 then shows this frame with a first-story column removed, representing the frame of the structure after an explosion or some event has caused a column loss. In reference to this project, no external forces from the triggering event are included in the analysis since these can be extremely hard to predict.

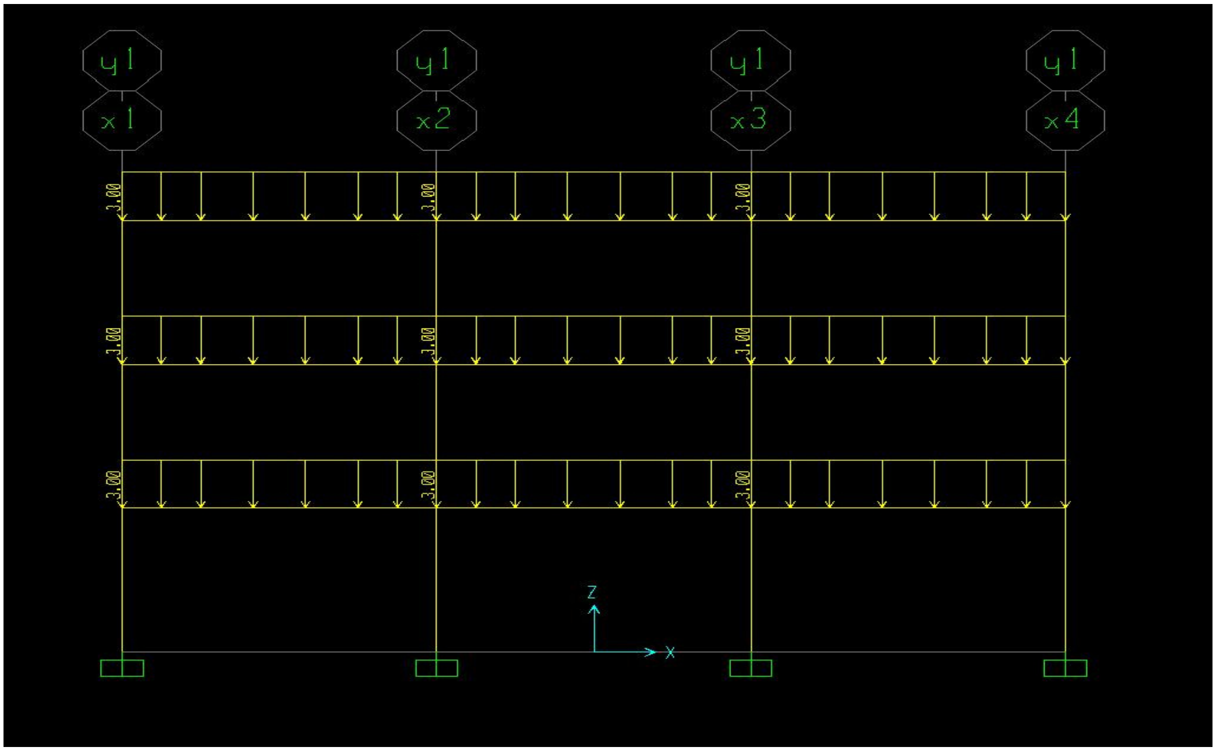

Figure 3.2: SAP2000 frame before column loss

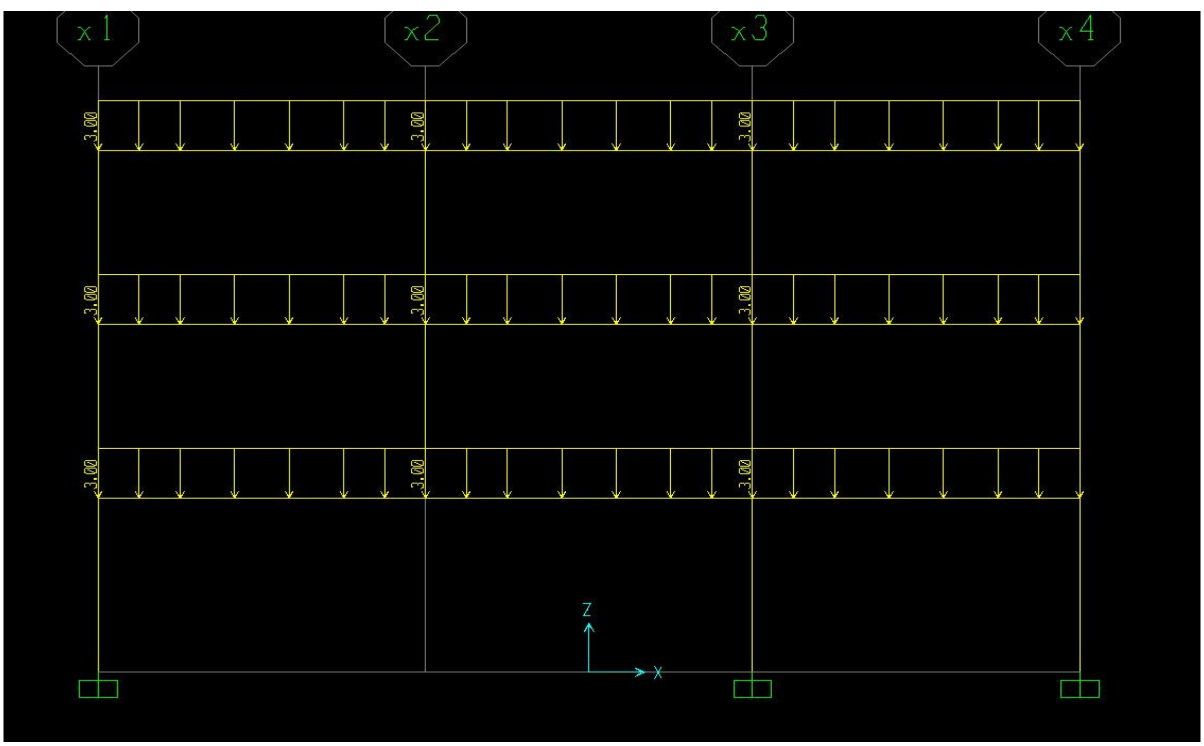

Figure 3.3: SAP2000 frame after column loss

The idea is to analyze the frame and study the moment diagrams in the part of the beam directly above where the column loss occurred. Typically that joint region would have steel in the upper section of the beam to account for the negative moment near the column faces and would have minimal or no required reinforcing steel in the lower part of the cross section of the beam where overlapped steel exists. The problem addressed by this project can be seen in Figure 3.4 after the column is removed. As soon as support from the lower column is removed, the moment in the joint region becomes a very large positive value and would easily cause the beam to fail if the minimal reinforcing steel was all that was present in the lower section of the beam at the beam-column joint location. Figure 3.4 below shows the cross section of the beam and the reinforcing steel of concern to this study.

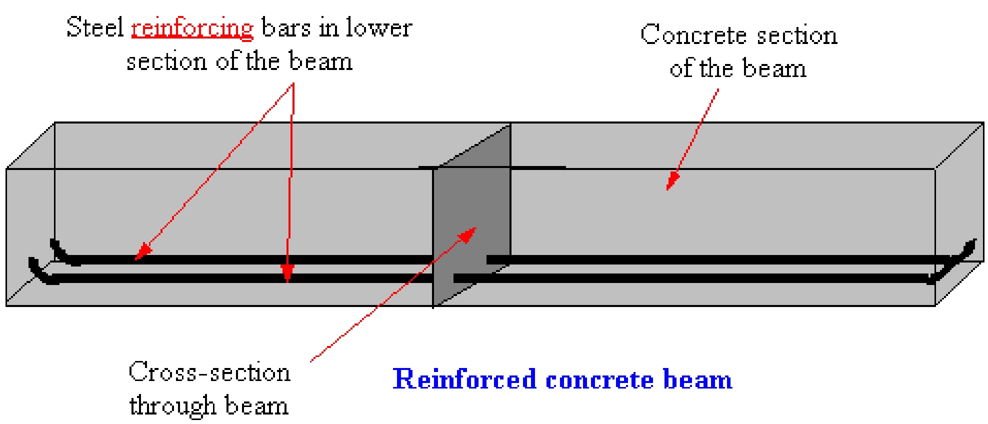

Figure 3.4: Beam cross section and reinforcing steel for positive moment

Figures 3.5 and 3.6 show the change in the moment diagrams below. In this frame example, Figure 3.5 shows that the beam has a relatively small negative moment under gravity loading when the column is in place. Figure 3.6 shows that when the column is lost much larger, almost twenty times in magnitude, values of a positive moment are reached in the same beam-column joint location.

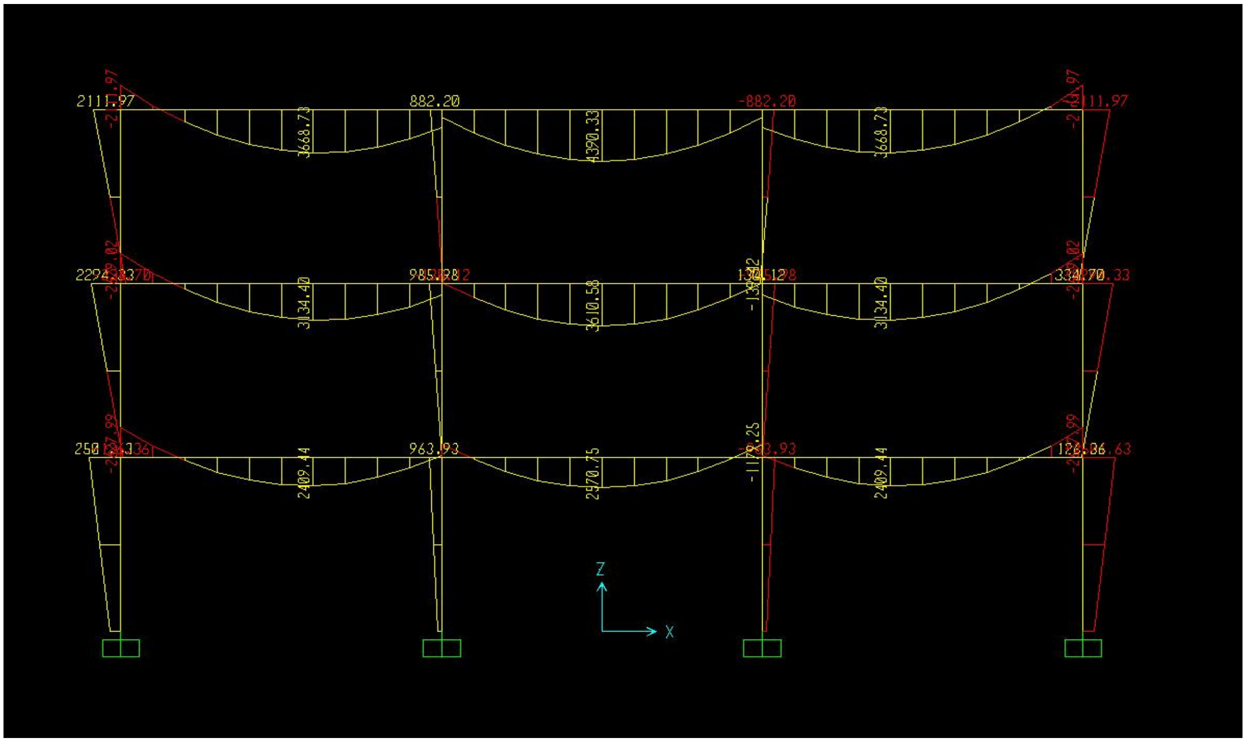

Figure 3.5: SAP2000 loaded frame with moment diagram before column loss

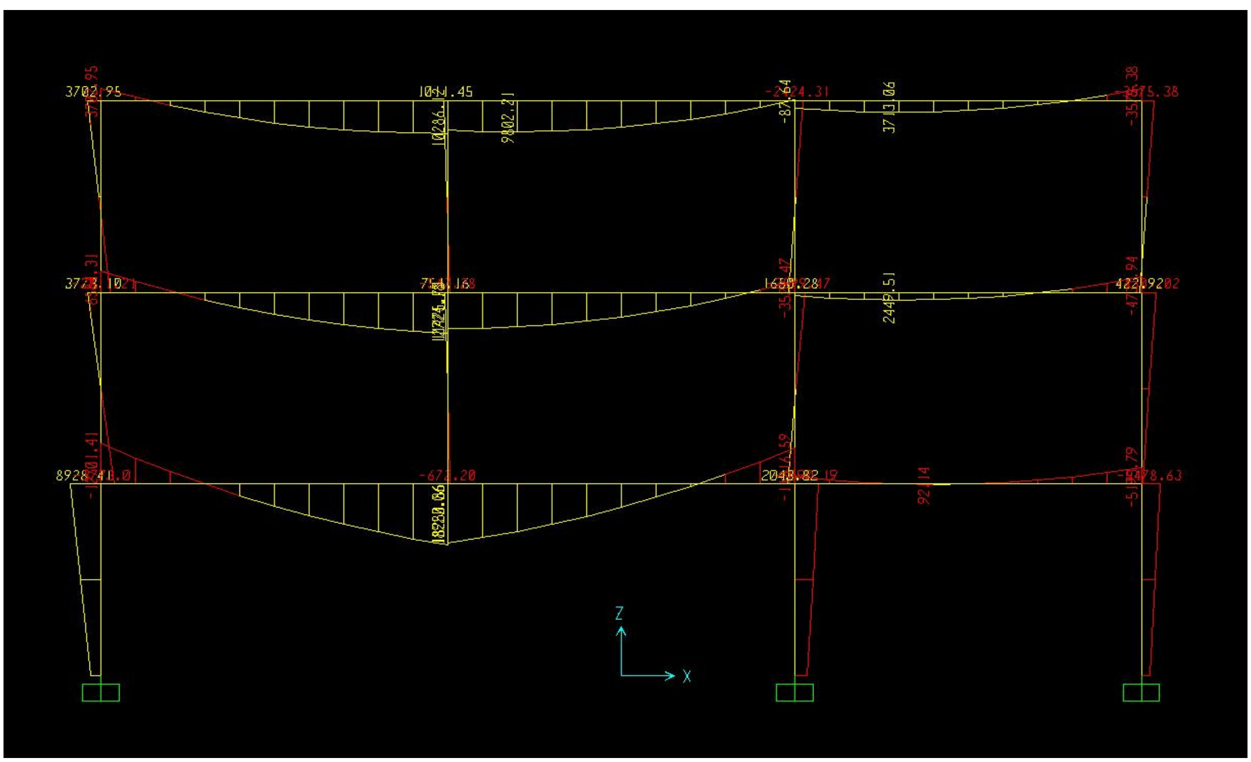

Figure 3.6: SAP2000 loaded frame with moment diagram after column loss

3.3 Research Significance and Needs

This phenomenon of progressive collapse is typically not considered in structural design, but in the rare event that it would occur, it can easily be seen how devastating the effects could be to a structure. Once beam failure occurs, the frame enters the conditions described previously for progressive collapse and large amounts of damage will result which could present a major safety hazard for those people in or near the structure. With the problem explicitly shown, the purpose of this project can better be explained.

All calculations and analysis for the remainder of this project are geared towards determining how to prevent the failure of the continuous beam when column loss occurs and the moment demand changes drastically as shown above. With this idea in mind, it can be seen that establishing a minimum area of steel to be present in the lower section of the beam would be the ideal way to prevent collapse of the beam. Many variables exist that have an effect on the required amount of steel needed in the continuous beam to prevent its collapse. For this reason, several different computer programs are developed to cover the widest range of variables possible. These variables include span length between columns, beam dimensions, material properties, uniform gravity loading values, and the magnitude of axial force in the column acting above the span where column loss had occurred. These factors are all incorporated and varied within the computer models in combination with a few different analysis methods.

Two ways the analysis can be performed for the beams would be to consider only their normal yielding moment capacities or to consider their plastic capacities assuming the proper confinement details were present to reach this capacity. Each of these methods was also then divided based upon the different potential end conditions at the beam and column joints. Moment distributions along the span of the beam were calculated in two ways. Pinned end moment conditions and fixed end moment conditions are the major extremes in terms of how joints behave in a frame. By analyzing both types of end conditions, a better idea of where the real results fall can be determined. Pinned end conditions produce higher moments and thus conservative amounts of steel required. Fixed end conditions are much closer to the reality of joint behavior although they do not tend to act solely in this manner. Therefore the fixed end moment programs should be closer to real behavior but have a lack of conservativeness due to real behavior in frames and joints. Figures 3.7 and 3.8 below give a visual representation of the different types of analysis built into the programs to obtain certain ranges of results.

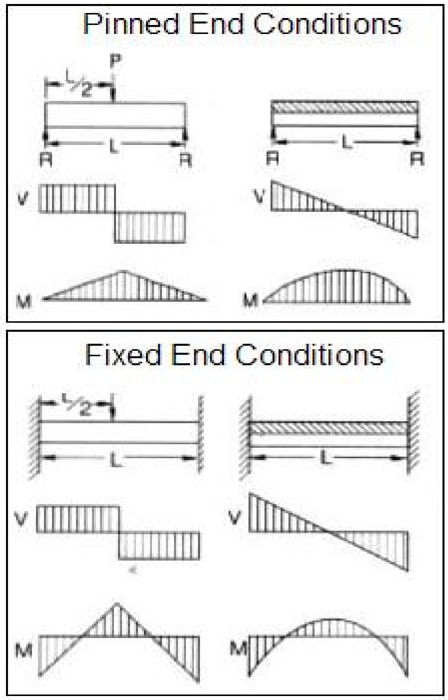

Figure 3.7: Pinned end conditions versus fixed end conditions

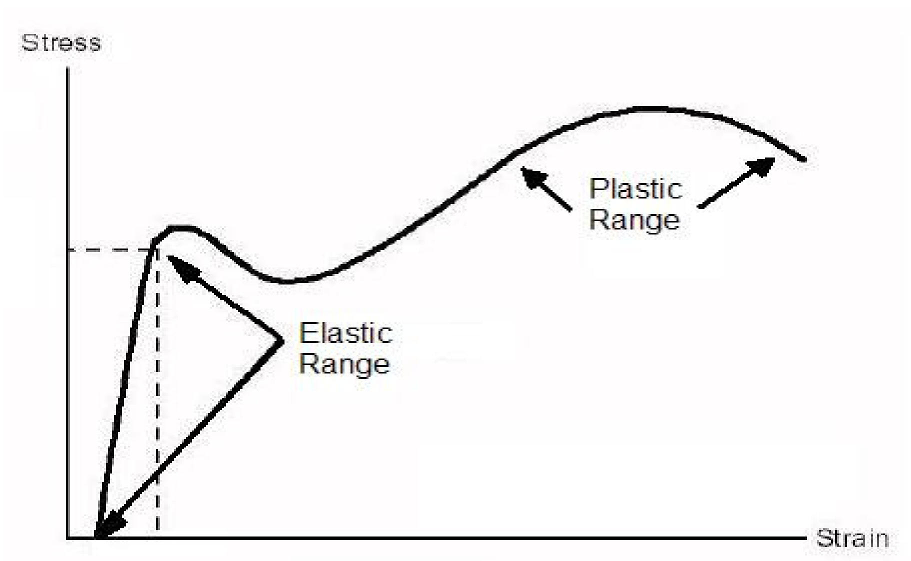

Figure 3.8: Nominal elastic moment capacity versus plastic moment capacity

CHAPTER 4

PROPERTIES OF MATERIALS AND MODEL FOR STRUCTURE

4.1 Introduction

The materials used in this project shall be those representing a typical low-rise reinforced concrete building located in high or low seismic regions. Specific structural design materials consist of reinforced concrete made from normal weight, normal strength concrete as well as normal strength steel rebar. Concrete strength may be input into the model at different desired values and would normally be used around 4000 psi. The steel strength may also be input into the model at different values which would typically be around 60 ksi for yielding and 100 ksi. The modulus of elasticity of steel is taken as 29,000 ksi for linear elastic behavior.

Multiple programs are used to model the behavior of a beam under various conditions that could cause progressive collapse. All programs developed in this study and the beam designs meet the requirements of ACI 318 design code (2008). These equations are discussed in greater detail later in this chapter. As previously described, four variations of analysis are developed to obtain results that are compared upon completion of analysis. These variations include differences in moment strength capacity of the beam and the end conditions affecting moment distributions across the length of the beam. Nominal moment strength, plastic moment strength, pinned end support conditions, and fixed end support conditions are combined in different ways to obtain a range of possible resulting data. Ultimately, the amount of continuous steel required through the bottom of the beam section near the center column joints in a beam are calculated for various loading conditions. All models and programs are based upon the simplified setup shown in Figure 3.1.

4.2 Development of the Model

Basic analysis involves modeling of a continuous beam over the length of two spans in a frame. This beam has different end conditions at the column supports based upon the program being used. The center of the continuous beam represents the location of the lost column. The purpose of this study is to perform analysis for a progressive collapse type situation and therefore the column underneath the center of the beam will be nonexistent, as if it had been lost due to an explosion. However, the manner in which the column was lost is assumed irrelevant and no outside forces are applied to the system. This model assumption works in accordance with the alternate path method where triggering events do not affect the system. With this type of model setup, moment equations can be derived for the center of the beam based on the given length, the magnitude of distributed loads along the span, and value of the concentrated load pressing on the center of the beam from the column above. The purpose of this setup is so that ultimately the required amount of continuous steel needed through column connections can be directly calculated for any range of uniform or concentrated load values and for any beam dimensions being used. User of the program or this method should be able to use SAP2000 or other methods by hand to calculate the magnitude of force in the column above the beam and input this value into the program along with the physical dimensions in existence to obtain a value for required continuous steel through the lower section of the beam through the beam-column joint area. This analysis assumes the appropriate amount of transverse steel has been used to prevent shear failure.

The setup of the model was determined deliberately to simplify the resulting data. Symmetric dimensions and loading conditions help to provide consistent results where the maximum moment values and corresponding minimum values of steel always fall directly in the center of the beam between its two outer column supports. This slightly simplifies the work because the required steel at midspan of the two-span continuous beam is the desired value since that is where the column load is acting and where continuous steel in the beam-column joint needs to be calculated.

4.3 Specifics of Programs

Factors that can be modified in the programs include dimensions of the beam, length of the span between each column, strength properties of the concrete and the steel rebar, as well as the loading conditions. Beam dimensions can easily be modified in order to allow use for engineers to check their designs to ensure they have adequate continuous steel amounts running through their column joints. This experiment uses constant values for the beam dimensions to keep consistent results.

The length of the span can be changed with the same purpose that the beam dimensions can be changed. Engineers can input the span length they have to check if they have the required amount of steel. The span length is also held constant for each test during this experiment to simplify results. Different tests can then be represented on different graphs or tables in order to collect results for numerous cases. Note this span length represents that between each column through the frame. The beam being analyzed then has a span length twice that amount due to the situation being modeled where the lower middle support column is removed.

The strength properties of concrete and steel can be modified to check the differences in continuous steel required for different cases. This also allows engineers to check their current design, whatever it may be. Modifiable strength properties include the compressive strength of the concrete, $f_c^{'}$, as well as the yielding stress of steel, f_y, and the ultimate stress in the steel, f_u. As mentioned before, loading conditions may be found by simply hand calculating the concentrated force acting on the center of the beam or by using SAP2000 to input distributed loads throughout the frame.

4.4 Moment Capacity Equations

All programs are based generally upon assumptions used in the ACI 318 design code (2008). This was done for simplification purposes and because the design codes are what determines real design. Strain hardening of the reinforcement and confinement effects produced by closely spaced transverse steel are not included directly in the calculations. Compared to nominal moment strength (M_n), based on yield strength of steel, the plastic moment calculations do add to the moment capacity based upon the idea of confinement. No direct strain calculations are performed to determine the exact strength at any time. Rather, the recommendation by ACI of increasing the yield strength of the steel by 25 percent is utilized in the plastic moment capacity equations (M_p). This should produce results for the two plastic moment capacity programs much closer to actual behavior, assuming they contain the proper shear reinforcement required to produce plastic moments, although those results are less conservative. Details of the capacity equations used in the calculations are presented below.

$$\phi M_n = A_s f_y \left(d - \frac{a}{2} \right) \left(\frac{1ft}{12in.} \right) \geq M_u \qquad \phi M_p = A_s \left(1.25 f_y \right) \left(d - \frac{a}{2} \right) \geq M_u$$

Where the variables are defined as:

ϕM_n: The calculated nominal moment capacity of the beam, [$k \cdot ft$]

ϕM_p: The calculated plastic moment capacity of the beam, [$k \cdot ft$]

A_s: The amount of tension reinforcing steel, [$in.^2$]

f_y: The yielding strength of steel, f_y=60[ksi]

d: The distance from extreme compression zone to the centroid of tension steel, d=20.5[$in.$]

a: The transformed width of the compression block, $a = \dfrac{A_s f_y}{0.85 f_c^{'} b}, \dfrac{A_s \left(1.25 f_y \right)}{0.85 f_c^{'} b}$ [$in.$]

$f_c^{'}$: The compressive strength of concrete, $f_c^{'}$=4000[psi]

b: The width of the beam, b=20[$in.$]

M_u: The moment demand placed on the beam from loading conditions, [$k \cdot ft$]

The differences in programs based upon the moment distributions affects the initial parts of the programs where the shear and moment diagrams across the entire length of the beam are calculated. These changes correspondingly only affect the moment distribution and maximum positive moment values. This does not affect the capacity equations in any way although they do result in different amounts of steel required to due unlike maximum moments used to determine of required amounts of steel. All models shall be presented in full throughout the study.

CHAPTER 5

IMPLEMENTATION AND ASSUMPTIONS IN MODELING AND ANALYSIS

5.1 Introduction

A wide variety of variables are present in the actual design process for structural members in reinforced concrete frame systems. Properties of concrete and steel, beam dimensions, among many other details and assumptions of analysis are presented and explained in this chapter.

The models developed and used in this study were created as programs in MATLAB and all further analysis and results are obtained through the use of these programs. While an actual user may need to use SAP2000 or other programs to obtain critical input data, the scope of this project is that a large range of forces are tested where the force applied to the center of a continuous beam across a double-span is the main variable. Choosing all other variables present with the situation to remain constant while only changing this force acting on midspan, effectively representing the loading conditions, gives a clear relation between loading conditions and the amount of continuous steel required to prevent progressive collapse. Factors to remain constant through all analysis include all of the properties of the concrete, the steel, and all dimensional properties of the beam. Span length is varied but each test consists of a single span length that is specified for reference. A comprehensive list of assumptions incorporated into the program design is also given in order to show the restrictions and limitations of its usage.

5.2 Concrete Properties

The concrete has several properties that are held constant through this study. The first of these which may also be the most influential to the results is the ultimate compressive strength of the concrete, $f_c^{'}$. This property is held at a constant value of 4000 psi. This value is chosen because it is a common concrete strength used in structural design of reinforced concrete members. The crushing strain of concrete would typically be another important factor in determination of beam strength but since only the ACI equations are used, this value is not included in the programs. This concrete strain for the standard yield or nominal moment strength would be at a value of 0.003 and then a higher value for plastic moment strength can be specified depending on the transverse reinforcement spacing. This idea of a varying crushing strain for plastic moment analysis was one reason that detailed stress and strain analysis was not performed. Not only would the programs have to be extremely robust but the data would become even less conservative and not as useful as the results found by using the suggested ACI equation. Also, as standard design and analysis procedures work, the tensile strength of concrete is ignored. The value of concrete tensile strength is relatively low and by ignoring it there is a small additional amount of conservatism built into the analysis.

5.3 Steel Properties

Just as with concrete, all of the properties for steel remain at constant values throughout the study. The most important of these values includes the yield and ultimate stress of the steel which is held at 60 ksi and 100 ksi, respectively. The programs do not actually use the rupture strength of the steel once again as that is not part of the ACI equation previously shown in Chapter 4. The assumed failure or limit value of the moment capacity equation is determined by the parabolic nature of the equation and results end once that equation peaks out at a certain value of steel and causes the equation to no longer be applicable. The modulus of elasticity is taken as 29,000 ksi which is a very common value used for steel reinforcement. These previous factors affect the results of all models. The following constants are can be useful but do not affect the programs written for this study. The rupture strain of steel can be taken at a value of 0.12. Also the steel strain for ultimate strength can be set equal to a value of 0.06. The strain hardening percentage of steel can vary but could be taken at a value of 2% which would show a large but reasonable amount of difference between the results obtained from the ACI code versus those found by modeling actual behavior.

5.4 Beam Dimensions

Several dimensions are held at constant values for parts of this project. The height and width of the beam are modeled as 24 and 20 inches, respectively. These dimensions were chosen because they are reasonable beam size dimensions and because a larger width better accommodates the assumption of steel placement within the beam which is mentioned later in this chapter. The span length between each column is another dimension that is held at a constant value. However, figures are developed with analytical data for several different span lengths. A wide range of span lengths were chosen because different spans better show sensitivity to the effects of the force at midspan from the column above. Thus varying span lengths better show a relation between loading and steel required.

5.5 Other Modeling and Design Assumptions

Many assumptions are made in order to simplify the development of the program and obtain useful consistent results. Assumptions of the model include steel placement, shear strength, end-reactions of the beam, contributions of compression steel, two-dimensional modeling, external forces, and strength reduction factors. These assumptions affect both models in essentially the same manner.

5.5.1 Steel Placement

The assumption involving steel placement is that all reinforcement steel contributing to moment capacity is located in the lower part of the beam section at a distance 'd' below the top of the section. The value of 'd' is set equal to 20.5 in. which is assumed to be 3.5 in. less than the height of the beam in all cases. The reason for this assumption is to greatly simplify the calculations needed to determine if a certain amount of steel can fit in one row or two and what the new corresponding 'd' value would be for the beam. The distance of 3.5 in. from the bottom of the beam accounts for the 1.5 in. of clear cover concrete, 0.5 in. for maximum stirrup thickness, and an additional 1.5 in. to reach the centroid of reinforcement steel. That last 1.5 in. is an estimate of how far above the edge of the stirrup that the centroid of steel would be with two rows of reinforcement needed. This was determined to be necessary due to the limited amount of steel that can be placed within a single row in the beam being modeled. As it could be determined from Table 5.1 below, the largest amount of steel to fit in one row in this beam is

9.36 $in.^2$ of steel composed of 6 #11 bars. Thus all results requiring more steel than this would have to be placed in two rows. Then with two available rows 18.72 $in.^2$ of steel could be used. Most of the results fall below this value of steel area and would have a centroid nearly located as described above. This helps produce conservative results when all of the steel could be placed in a single row closer to the bottom of the beam while preventing an overestimation of moment capacity in cases where two rows of steel would actually be needed.

Table 5.1: Minimum Required Beam Widths (in.), (Limbrunner, 2007)

Number of bars in one layer	Bar Size							
	#4	#5	#6	#7	#8	#9	#10	#11
2	6	6	6.5	6.5	7	7.5	8	8
3	7.5	8	8	8.5	9	9.5	10.5	11
4	9	9.5	10	10.5	11	12	13	14
5	10.5	11	11.5	12.5	13	14	15.5	16.5
6	12	12.5	13.5	14	15	16.5	18	19.5
7	13.5	14.5	15	16	17	18.5	20.5	22.5
8	15	16	17	18	19	21	23	25
9	16.5	17.5	18.5	20	21	23	25.5	28
10	18	19	20.5	21.5	23	25.5	28	31

5.5.2 Shear Capacity

The next assumption is simply that the beam would be designed to adequately handle the shear demand placed upon it. While realistically the direct concentrated force from the column above the beam or the beam-column joint areas may cause problems with shear, the focus of this study is on the steel required for moment capacity and thus it is be assumed that adequate stirrups for shear design are provided. Also, in the cases where plastic moment strength is being used, it is assumed that adequate transverse reinforcement is present in order to develop the plastic hinge in the beam. This applies in high-seismic regions where this type of design would be required by the codes.

5.5.3 End Conditions

One current inaccuracy with the program is that the exact end conditions are not known for the beam in a given frame. Continuous beams in a frame system such as the one presented would likely exhibit end reactions of a fixed nature, however this is not always the exact case. Frame systems are very complicated and different elements react with each other to produce different results in moment distributions. With this said, two major extremes are analyzed to get a range of how the system could act. Pinned end conditions produce higher maximum positive moments while fixed end supports distribute the moment throughout the beam and lower the maximum value. Since real cases should all fall in between these two extreme situations, a range of possibilities are covered. Engineers with enough experience or the ability to predict stiffness factors in the frames could decide upon an interpolation factor between the two cases in order to best estimate the actual situation they would be dealing with. Also to be noted in this discussion, it is possible that the negative moments acting on the far ends of the span could be much higher than before after column removal occurs. Determining steel amount and detailing in those areas is not in the scope of this project and would have to be studied separately with other programs in order to prevent failure at the joints on the ends. This project only focuses on providing enough steel in the center joint where column loss has occurred in order to prevent failure at that point.

5.5.4 Compression Steel

Compression steel is not included in the calculations of moment capacity. This is assumed because it would require detailed calculations to determine the amount of steel in the top section of the beam based on each individual loading case. It is also difficult to predict the actual amount of compression steel placed in the beam as it changes with every situation. Thus it would be more practical and more conservative to ignore any contribution of compression steel in moment calculations.

5.5.5 Two Dimensional Frame

In this research a two-dimensional frame is chosen because all of the interacting effects involved with a three-dimensional model are extremely complicated and difficult to represent in a program for every case. Ignoring the three-dimensional effects is a conservative assumption because the other beams connected to the joint where column loss occurred could help transfer the loads to other parts of the structure. This could decrease the amount of moment needed to be carried by the continuous beam being analyzed but the immense uncertainty in magnitude of this effect makes it too difficult to include in the programs and helps to produce more conservative results in the end.

5.5.6 External Forces

This assumption is consistent with the alternate path method in that the triggering event is irrelevant and has no direct effect on the beam or frame system. It would be like having an immediate column removal with no forces acting on the system. This is acceptable because any forces introduced to the system are completely unknown and the alternate path method is a very common mode of threat-independent analysis for progressive collapse situations.

5.5.7 Strength Reduction Factors

This study assumes the strength reduction factor, ϕ, has a constant value of 0.9. This is not always the true case as the factor is dependant on the internal strains within the beam. Since these strains were not calculated, the assumption that steel would always be yielding was made in order to simplify the calculation process. This is not a conservative assumption but it does not likely have a major bearing on the results since the factor in general is simply an added safety factor for uncertainty.

CHAPTER 6

PRESENTATION OF ANALYSIS RESULTS

6.1 Introduction

With all of the assumptions and variables listed and described, the different programs that were written for the various modes of analysis were run and tested. Shear and moment diagrams as well as minimum steel reinforcement diagrams and maximum column loading tables are developed and presented throughout this chapter to show these results.

The programs containing the ACI calculations were tested with a set of constant values for beam dimensions and material properties. The continuous beam was modeled with a width of 20 in. and a height of 24 in. where the distance from the extreme compression zone to the centroid of tension steel, effective depth 'd', is assumed to be 20.5 in. The material properties of the modeled concrete included the compressive strength which was taken as 4000 psi. The material properties of the modeled steel were consistent with those of Grade 60 steel where the yield strength is 60 ksi and the ultimate strength was 100 ksi. The major variable was the concentrated force acting at the midspan of the beam from the center column load above the beam. Other variables tested and separated within the results included the magnitude of distributed loads and the length of the span between columns. The required minimum amount of steel needed to prevent flexural failure of the beam at the column joint location was then with respect to the load variable and other factors such as span length and uniform load acting directly on the beam itself.

6.2 Shear and Moment Diagrams

Shear and moment values are calculated within the program for every point along the beam and the results are plotted for each. These calculations are required in order to obtain the moment demand at the midspan of the beam which happens to coincide with the maximum positive moment value due to the symmetrical setup of the model. In some situations the negative moments near the ends of the beam have a higher magnitude than the positive moment at the midspan of the beam. The negative moments at the end locations were not considered as part of the scope of this study and it was assumed that adequate capacity was present to account for the negative moment demands in those locations. The shear and moment diagrams are properly adjusted for fixed end conditions using the principle of superposition for concentrated and uniformly distributed loads acting symmetrically on the beam.

Figures 6.1 and 6.2 show examples of the shear and moment distribution across the beam for different end conditions. One special case, in Figure 6.1, shows a beam where the span length is 30 feet between columns, the uniform distributed load is 2 kips/foot, the column load is 50 kips, and the beam has pinned end conditions. The other case, in Figure 6.2, shows the same beam with fixed end conditions.

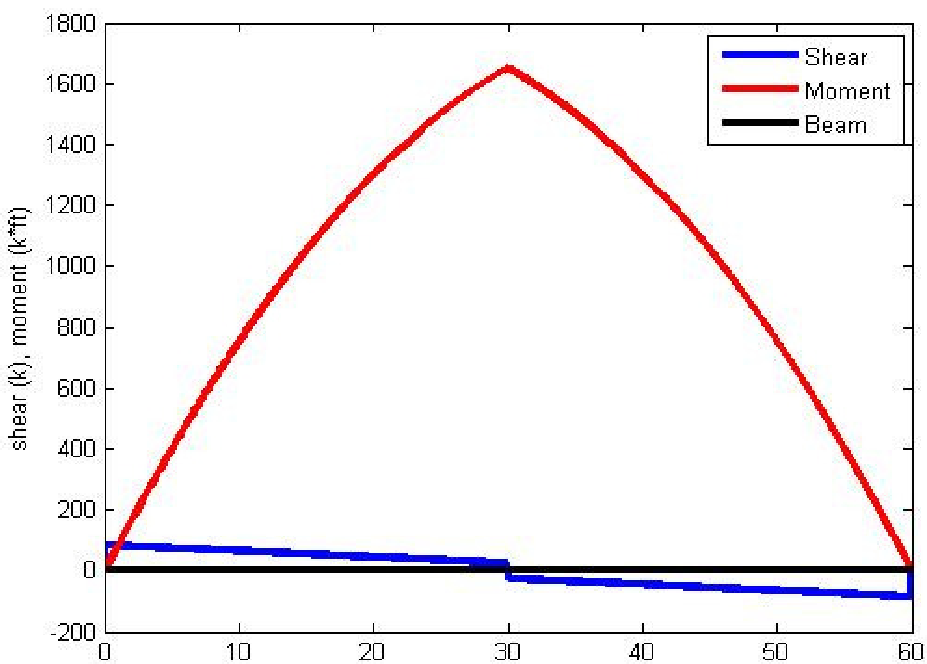

Figure 6.1: Shear and moment diagrams for pinned end condition

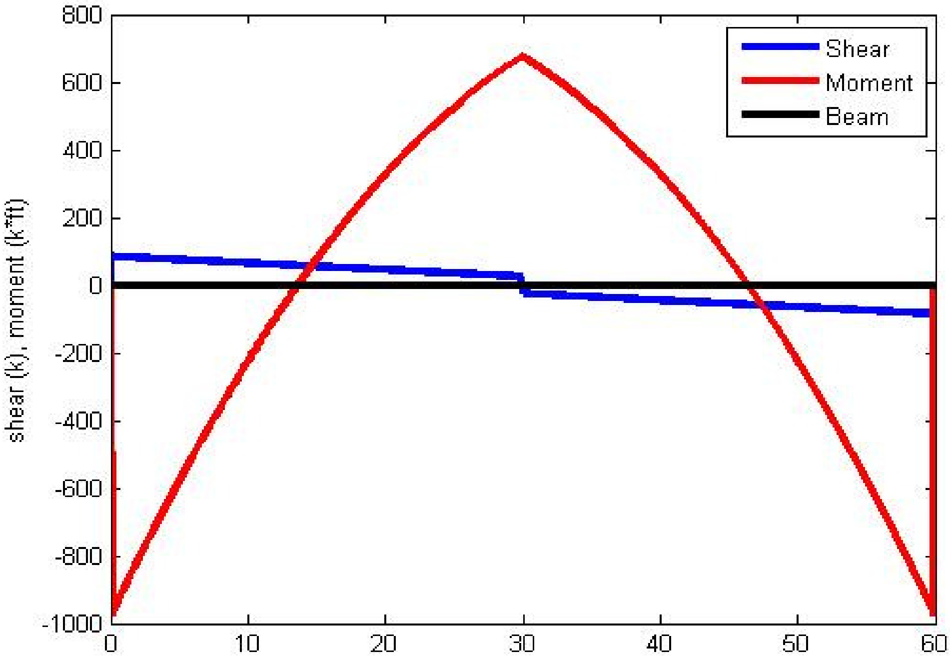

Figure 6.2: Shear and moment diagrams for fixed end condition

6.3 Minimum Steel Reinforcement

One of the ultimate goals of this study is to obtain a relationship between the amount of steel required to prevent progressive collapse as defined in this study and all loading conditions and dimensions. The results are obtained through the programs by using the calculated moment diagrams along with the appropriate capacity equation for the beam being analyzed. These results are included in Appendices A through D and each shows the relationship between the column load and the required minimum steel at a specific span length with specified end conditions and moment capacity equations.

Figure 6.3 shows one example of these results as described above for a span length of 10 feet with pinned end conditions and nominal moment strength. The x-axis on the plots represents the column load measured in kips. The y-axis on the plots represents the required minimum steel at the midspan of the beam in square inches. Each curve on the graph represents the results with all values remaining the same except for the uniform load applied directly to the beam. The curves end at the point where the capacity equation no longer applies, meaning that the beam cannot carry the load regardless of the amount of steel put in the beam due to the assumptions made in the formulation of the ACI equations.

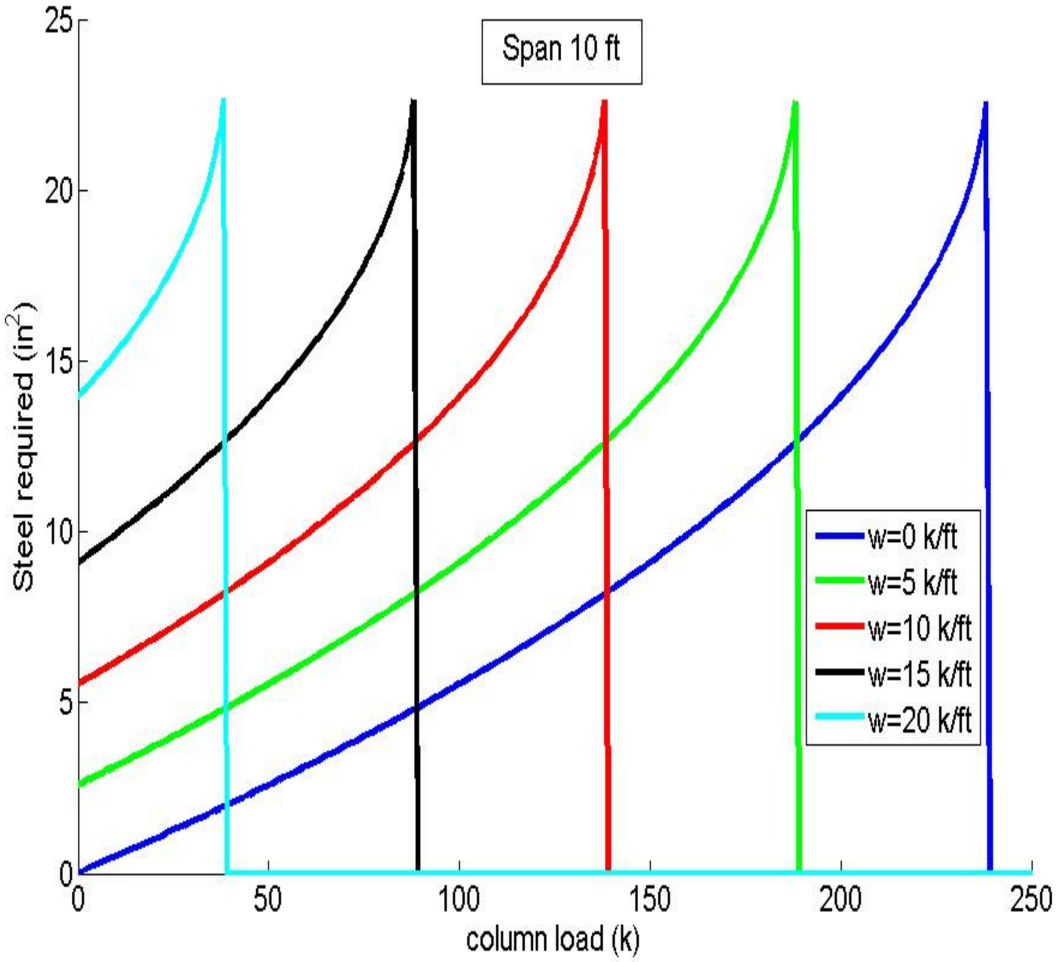

Figure 6.3: Required steel with pinned/nominal conditions with a 10ft span

6.4 Maximum Column Loading

Tables 6.1 through 6.4 are quick reference of the maximum column load values allowed for different span lengths, uniform loading conditions, end reaction conditions, and types of moment strength capacity. The tables show the maximum load values in the column that cause the ACI equation to break down and no longer apply. This occurs in the minimum steel reinforcement diagrams at the points where the lines suddenly end and drop off the graph. This occurs due to the assumptions made with the formulation of the capacity equations. This means that for the given conditions, any concentrated column load higher than those shown in the tables can result in collapse of the beam and lead to progressive collapse. No amount of steel according to the equation would be able to develop enough moment capacity to carry the loads in such a situation. Notice that the results for nominal moment capacity and plastic moment capacity have the exact same values. This is because longitudinal steel yielding is selected as the limiting condition to calculate the moment strength. It can be seen in the graphs in the appendices that the steel required for each case differs.

Table 6.1: Max. column load values for pinned end / nominal conditions

End Cond./Capacity	Maximum Allowable Column Loads (kips)									
Pinned/Nominal	**Uniform Distributed Load (k/ft)**									
Span Length (ft)	**0**	**1**	**2**	**4**	**6**	**8**	**10**	**12**	**15**	**20**
10	238	228	218	198	178	158	138	118	88	38
15	158	143	128	98	68	38	8	0	0	0
20	119	99	79	39	0	0	0	0	0	0
30	79	49	19	0	0	0	0	0	0	0
40	59	19	0	0	0	0	0	0	0	0
50	47	0	0	0	0	0	0	0	0	0

Table 6.2: Max. column load values for pinned end / plastic conditions

End Cond./Capacity	Maximum Allowable Column Loads (kips)									
Pinned/Plastic	**Uniform Distributed Load (k/ft)**									
Span Length (ft)	**0**	**1**	**2**	**4**	**6**	**8**	**10**	**12**	**15**	**20**
10	238	228	218	198	178	158	138	118	88	38
15	158	143	128	98	68	38	8	0	0	0
20	119	99	79	39	0	0	0	0	0	0
30	79	49	19	0	0	0	0	0	0	0
40	59	19	0	0	0	0	0	0	0	0
50	47	0	0	0	0	0	0	0	0	0

Table 6.3: Max. column load values for fixed end / nominal conditions

End Cond./Capacity	Maximum Allowable Column Loads (kips)									
Fixed/Nominal	**Uniform Distributed Load (k/ft)**									
Span Length (ft)	**0**	**1**	**2**	**4**	**6**	**8**	**10**	**12**	**15**	**20**
10	476	469	463	449	436	423	409	396	376	343
15	317	307	297	277	257	237	217	197	167	117
20	238	224	211	184	158	131	104	78	38	0
30	158	138	118	78	38	0	0	0	0	0
40	119	92	65	12	0	0	0	0	0	0
50	95	61	28	0	0	0	0	0	0	0

Table 6.4: Max. column load values for fixed end / plastic conditions

End Cond./Capacity	Maximum Allowable Column Loads (kips)									
Fixed/Plastic	**Uniform Distributed Load (k/ft)**									
Span Length (ft)	**0**	**1**	**2**	**4**	**6**	**8**	**10**	**12**	**15**	**20**
10	476	469	463	449	436	423	409	396	376	343
15	317	307	297	277	257	237	217	197	167	117
20	238	224	211	184	158	131	104	78	38	0
30	158	138	118	78	38	0	0	0	0	0
40	119	92	65	12	0	0	0	0	0	0
50	95	61	28	0	0	0	0	0	0	0

CHAPTER 7

INTERPRETATION OF RESULTS AND LIMITATIONS

7.1 Introduction

The design code states that a minimum of two continuous bars must be present at all locations along the length of the continuous beam near the bottom surface of a beam in structures located in moderate to high seismic regions. The design of these structures is governed by the requirements of Chapter 21 of ACI 318. ACI 318 allows for longitudinal bottom bars to be discontinuous in the joint regions of structures located in the low seismic regions. These structures would have very limit or no progressive collapse resistance. While the bars that are present are completely determined by the size and number of bars used for flexure in the tension zone, the proposed amounts of steel are good approximations of amounts of steel to include continuously through the column joint area of the beam.

An extremely important point to be noted here is that these values are not to be used blindly for several reasons. One reason is that these values, although obtained from the ACI equation, have been calculated based upon several assumptions. Most of these assumptions should be considered conservative in nature, but there are also assumptions made that may not always be of this nature such as strength reduction factor. Another reason these values should not be blindly used is because many constant values were used which may be different in real life applications. Beam dimensions and material properties were chosen specifically for this project. Different values may be put into the program but the graphs and tables presented in this project are based solely upon those constant values chosen previously.

7.2 Brittle Failure

As could be concluded from the presented results, a maximum value of steel of around 22.5 square inches can be used to prevent progressive collapse from occurring under some conditions provided. This is obviously an extremely high amount of steel to put in a typical beam. For the beam modeled in this project, the steel ratio would be at about 4.69% which is considerably higher than the recommended design value of about 1.2%. This brings up the problem of having too much steel which could lead to brittle failure which is not acceptable. Brittle failure must be avoided as ductility is of key importance to having safety in design. Thus the balanced condition ratio of steel in the beam must be calculated to see what the maximum amount of steel is allowed before the brittle condition takes place. The equations for determining the balanced condition amount of steel are shown below.

$$\frac{0.003}{c} = \frac{\varepsilon_t}{d-c} \qquad\qquad 0.85 * f_c^{'} * a * b = A_s * f_y$$

Where the variables are defined as:

ε_t: The tensile strain in the longitudinal steel, ε_t=0.00207

c: The distance from extreme compression zone to the neutral axis from, [$in.$]

d: The distance from extreme compression zone to the centroid of tension steel, d=20.5[$in.$]

$f_c^{'}$: The compressive strength of concrete, $f_c^{'}$=4000[psi]

a: The transformed width of the compression block, $a = 0.85 * c$ [$in.$]

b: The width of the beam, b=20[$in.$]

A_s: The amount of tension reinforcing steel, [$in.^2$]

f_y: The yielding strength of steel, f_y=60[ksi]

Plugging in the values for the variables and combining the equations relating stress and strain above, the maximum amount of steel allowed before brittle failure occurs is $11.69\,in.^2$ for the nominal moment capacity of the beam and $9.35\,in.^2$ for the plastic moment capacity of the beam. Thus all of the graph data in the Appendices A through D ideally does not apply when steel amounts higher than these values occur for their respective conditions. A horizontal cutoff line limits those graphs at $11.69\,in.^2$ for the nominal moment capacity graphs and $9.35\,in.^2$ for the plastic moment capacity graphs because use of any more steel may result in brittle failure.

7.3 Minimum Steel

Another consideration with the safety of a design is the minimum amount of steel required. If too little steel is present, the beam can easily fail under initial loading conditions. The minimum amount of steel for the modeled beam is calculated by using the equation below.

$$A_{s,\min} = 0.0033bd = 0.0033*20*20.5 = 1.35 \;\; in.^2$$

Thus it could be shown that the real limits on steel must fall between, for this specific case, $1.35\,in.^2$ and the higher brittle failure limits mentioned before. As another warning, this is a very important balancing act that must be satisfied according to ACI design codes without exception. Final results may prove that prevention of progressive collapse may not be possible in certain areas within a structure's frame if the steel required to carry loads does not fall within the acceptable range of steel values shown. In this situation it may be required that the design moves forward without change or that design changes must be made to the beam in question.

7.4 Other Considerations

One idea to solve this problem may be to add bars of adequate development length in only the region of the center joint to provide the required amount of steel to develop positive moment capacity in that area. The plausibility of this idea would need to be investigated through experimental tests. This idea of limiting the amount of steel to be put in the joint area as described may be the largest shortcoming of this study since the results obtained may not always be applicable in actual buildings.

Interpolating values between lines graphed on the plots for different uniform loads or between separate plots for different span lengths should be done carefully. It was determined that the horizontal spacing between individual lines on any one specific plot in the appendices is approximately equal to the value of the difference of the uniform loads multiplied by the span length of the plot in use. Figures 7.1 and 7.2 do not represent linear behavior and would require different interpolation in order to obtain accurate results.

Figure 7.1 shows the relationship between the maximum allowable column load versus the span length where each line represents a different value for the direct uniform load acting on the beam. This figure represents the analysis for pinned end conditions with nominal or plastic moment strength. Figure 7.2 shows the same relationships for fixed end conditions. Figure 7.3 shows the relationship between maximum allowable column load versus the uniform load where each line represents a different value for the span length. Figures 7.3 and 7.4 are developed for pinned and fixed support conditions respectively.

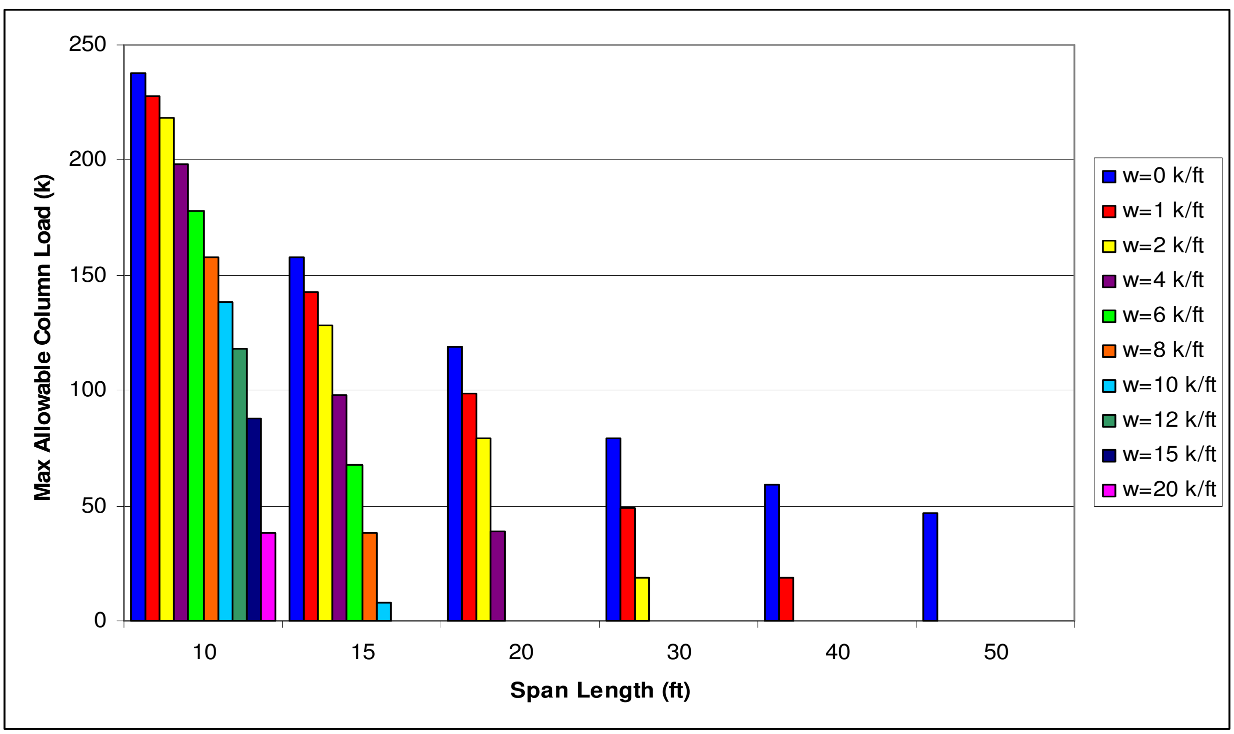

Figure 7.1: Max. allowable column load for pinned end conditions

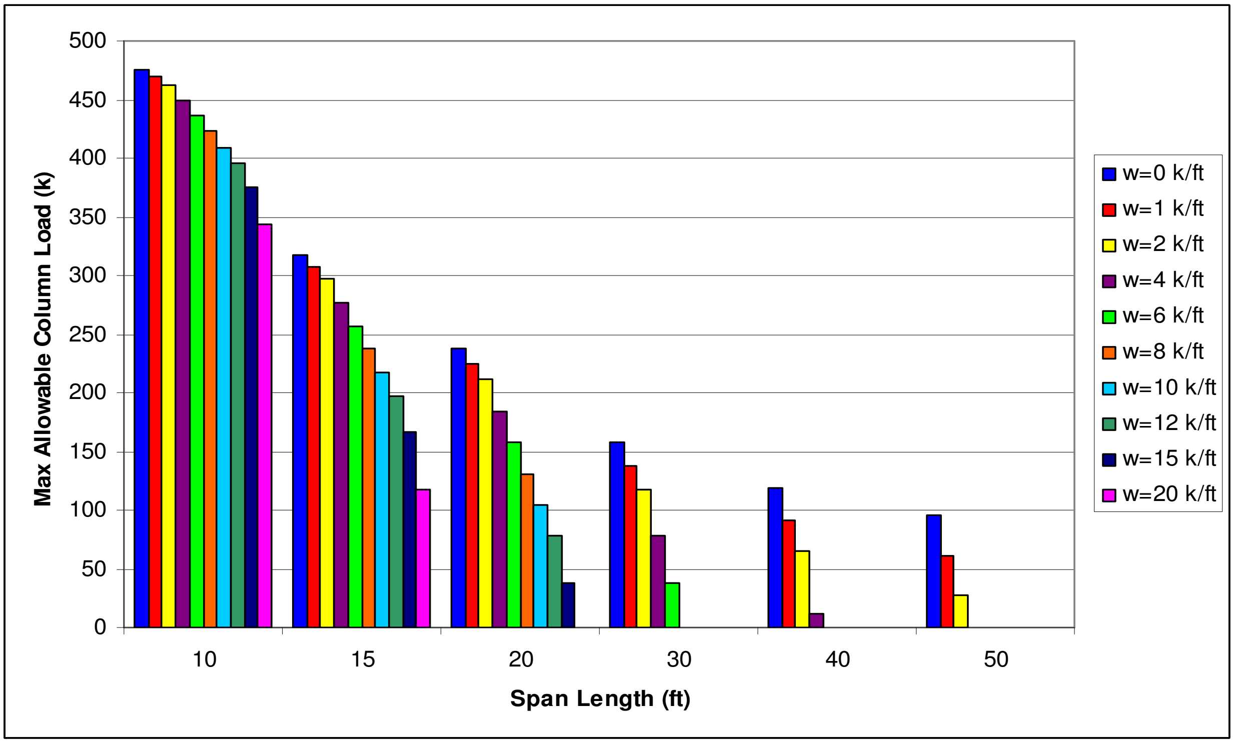

Figure 7.2: Max. allowable column loads for fixed end conditions

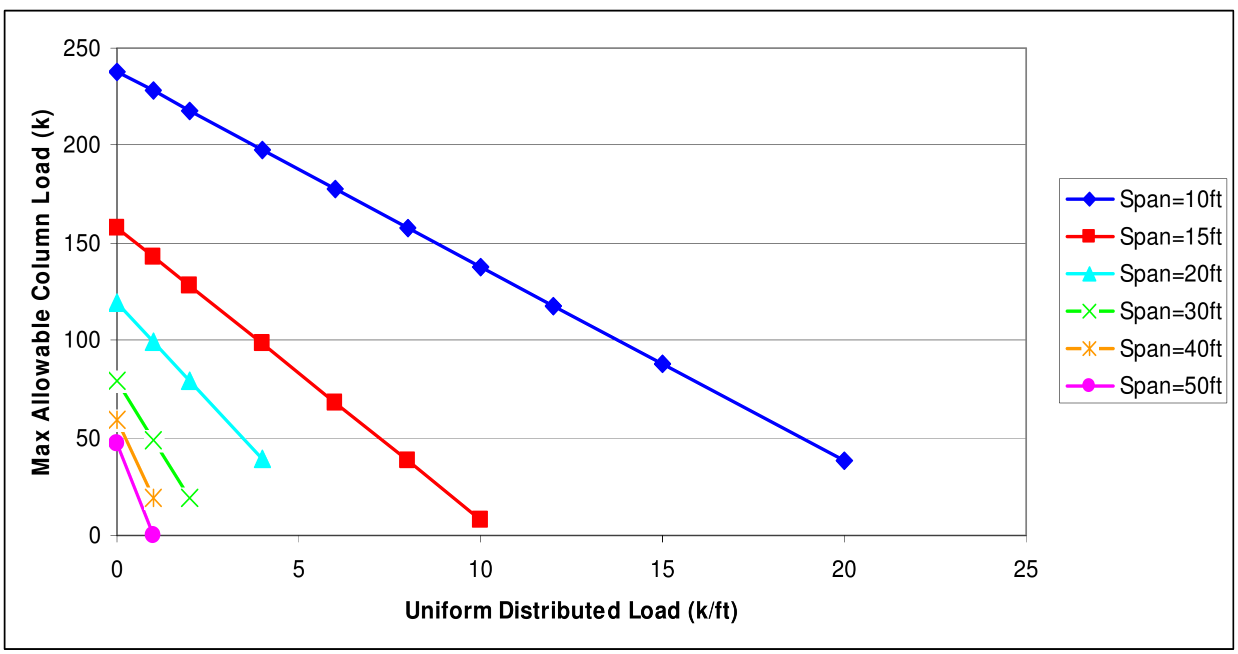

Figure 7.3: Max. allowable column loads for pinned end conditions

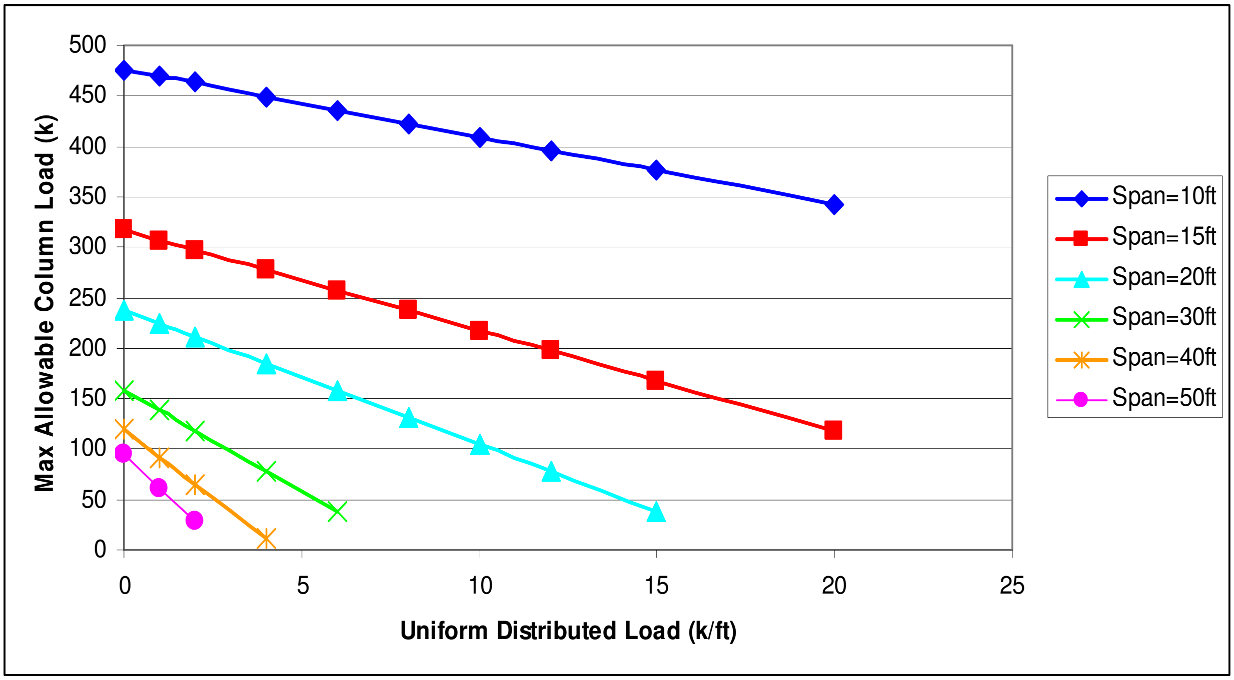

Figure 7.4: Max. allowable column loads for fixed end conditions

CHAPTER 8

SUMMARY AND CONCLUSION

8.1 Limitations of Research

While the values and results in the study are carefully obtained through general analysis and programming, they should be used with caution and as a guideline only at this time. As discussed in the previous chapter, many assumptions have been made in order to simplify processes and attain approximate results. While most of the assumptions have been determined to be conservative, they are not all explicitly of this manner. Not all assumptions may completely fall within ACI regulations and the results cannot be directly used as if they meet all code requirements. Thorough physical tests and experiments need to be used in order to confirm the accuracy and usefulness of the data obtained through this study.

With the limitations above being mentioned, the results obtained in this study should still prove to be good approximations and useful guidelines for analyzing progressive collapse sensitive situations. A generally good understanding of structural engineering is beneficial in determining the usefulness of this model to a particular situation in which it may be used for. Efforts were made to widen the applicable use of the results by means of testing many different combinations of span lengths and distributed loads with variable column loading until each beam reached failure. Also different methods of analysis including different end conditions and different beam moment capacities were used in combination to arrive at a range of results. This has given three complete degrees of freedom in the analysis where essentially only the beam dimensions and material properties are held constant. Increasing the beam dimensions or strength of the materials would ultimately result in a lower value of steel being needed while decreasing the dimensions or strength properties would result in more steel being needed in the column joint area to prevent progressive collapse in the beam.

The programs used for the modeling and analysis are included in Appendices E through H. These programs could be used for more specific purposes such as calculating the shear or moments at any point across the beam or for doing similar analysis with different beam dimensions or material property values. Most of the assumptions made previously may be altered or removed through user-modification of this program allowing for more accurate results. This helps to show the power of the developed program since it can be used in many ways not covered by the scope of this particular study.

8.2 Conclusions on Data

This study has gone into great depth and detail to obtain the results that have been presented. General formulations and assumptions were made whenever possible and the limitations of the programs and assumptions made have been thoroughly described. Many generalizations can be made based upon the results obtained in this study.

Several factors contribute to the minimum amount of steel required at the midspan of the continuous beam in order to prevent it from collapsing in the situation modeled in this study. Increasing span lengths, uniform loads, and axial column loads each increase the amount of steel required. While it is not shown in this study, through experience with the moment capacity equations, it can be assumed that a general increase in beam dimensions would decrease the amount of steel required for the same capacity.

Based upon the specific details of this study, it was found that the applicable range of steel to be used in the beam-column joint regions is between the minimum of $1.35\,in.^2$ and the maximum values of $11.69\,in.^2$ and $9.35\,in.^2$ for nominal moment capacity and plastic moment capacity analysis, respectively. The graphs in Appendices A through D show all values calculated according to the capacity equations with no consideration given to brittle or ductile failure conditions. For this reason is if of the highest importance that when using the program or results presented that the ratio of steel within the beam is below the balance condition in order to ensure brittle failure does not occur and safety is preserved.

Fixed end conditions provided directly lower moment demand values at the midspan of the beam when compared to the pinned end conditions. This resulted in less steel being needed for a specific loading condition. Ultimately, the fixed end condition analysis resulted in much higher values of concentrated and distributed loads being allowed for the same amount of steel as a pinned end condition case. Varying this portion of the analysis affected the demand side of the ACI equations directly causing horizontal shifts in the graphs of the minimum steel required.

Nominal moment capacity and plastic moment capacity analysis produced essentially the same results for a given loading, although the plastic moment capacity analysis required less steel for a given loading condition. This was due to the recommendation by ACI to simply increase the yielding strength of the steel by 25%. These changes affect the capacity side of the ACI equations directly causing vertical shifts in the graphs of the minimum steel required.

8.3 Summary and Goals

This project has aimed to give insight into the progressive collapse phenomena and give some guidelines into possible prevention of such an event. Maximum loading combinations and required amounts of steel for the location of the beam where column loading is applied have been provided in the various tables and figures, all of which are included in the appendices. This particular study has a few limitations but the information provided and the program used for the model analysis should prove to be quite useful in many ways towards solving problems involving progressive collapse in reinforced concrete structures. Users may need to combine some of their own calculations and work with this data to obtain final results, but this model seems to provide a wide range of data that can save engineers a lot of time and analysis if proven accurate. Future developments are encouraged to be made on this model and the programs used for it. Removal of some assumptions by the process of building a more in-depth and powerful program can only make the results more accurate.

The ultimate goal of this project was to provide data that would give insight in ways to help prevent progressive collapse from occurring in reinforced concrete structures. Providing detailed graphs and tables to use as a reference in possibly textbooks or in the design code manuals are the final product desired to be produced by this study. While it seems that the assumptions and judgment that has been used in this study are sound with the general ideas used in structural engineering, physical tests must be conducted to confirm or deny the accuracy of the theoretical data produced in this project. Sources of error or problems in the development of the programs may be discovered in this process and lead to important changes or limitations to the results. Only after tests have been conducted and the results have been perfected can these tables be considered for widespread use in practice but hopefully the steps taken in this study are the first critical parts to lead towards greater safety in design and prevention of progressive collapse failures in the future.

REFERENCES

Abruzzo, J., Matta, A., and Panariello, G. 2006. Study of mitigation strategies for progressive collapse of a reinforced concrete commercial building. *Journal of Performance of Constructed Facilities* 20, (4): 384-390.

ACI Committee 318. 2008. Building Code Requirements for Structural Concrete (ACI 318-08) American Concrete Institute, Famington Hills, MI

Bao, Y., Kunnath, S. K., El-Tawil, S., and Lew, H. S. 2008. Macromodel-based simulation of progressive collapse: RC frame structures. *Journal of Structural Engineering* 134, (7): 1079-1091.

Federal Emergency Management Agency (FEMA). 2002. World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations. *FEMA* 403, May 2002.

Federal Emergency Management Agency (FEMA). 1996. Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation. *FEMA* 277, August 1996.

Giriunas, K. A. 2009. *Progressive collapse analysis of an existing building [electronic resource]* Honors Thesis, Ohio State University.

Hayes Jr., J. R., Woodson, S. C., Pekelnicky, R. G., Poland, C. D., Corley, W. G., and Sozen, M. 2005. Can strengthening for earthquake improve blast and progressive collapse resistance? *Journal of Structural Engineering* 131, (8): 1157-1177.

Li, Z., and Shi, Y. 2008. Methods for progressive collapse analysis of building structures under blast and impact loads. *Transactions of Tianjin University* 14, (5): 329-339.

Limbrunner, G. and Aghayere, A. 2007. Reinforced Concrete Design: 484.

MATLAB. Version 2010. The Mathworks Inc., Natick, MA.

Mohamed, O. A. 2009. Assessment of progressive collapse potential in corner floor panels of reinforced concrete buildings. *Engineering Structures* 31, (3): 749-757.

Pearson, C. and Delatte, N. 2005. Ronan Point Apartment Tower Collapse and its Effect on Building Codes. *Journal of Performance of Constructed Facilities* 19, (2): 172-177.

Pekau, O. A., and Cui, Y. 2006. Progressive collapse simulation of precast panel shear walls during earthquakes. *Computers and Structures* 84, (5-6): 400-412.

SAP2000. Version 2010. Computers & Structures, Berkeley, CA.

Sasani, Mehrdad, and Kropelnicki, J. 2008. Progressive collapse analysis of an RC Structure. *Structural Design of Tall and Special Buildings* 17, (4): 757-771.

Talaat, Mohamed, and Mosalam, K. M. 2009. Modeling progressive collapse in reinforced concrete buildings using direct element removal. *Earthquake Engineering and Structural Dynamics* 38, (5): 609-634.

Tsai, Meng-Hao, and Bing-Hui, L. 2008. Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure. *Engineering Structures* 30, (12): 3619-3628.

APPENDIX A

PINNED END / NOMINAL RESULTS

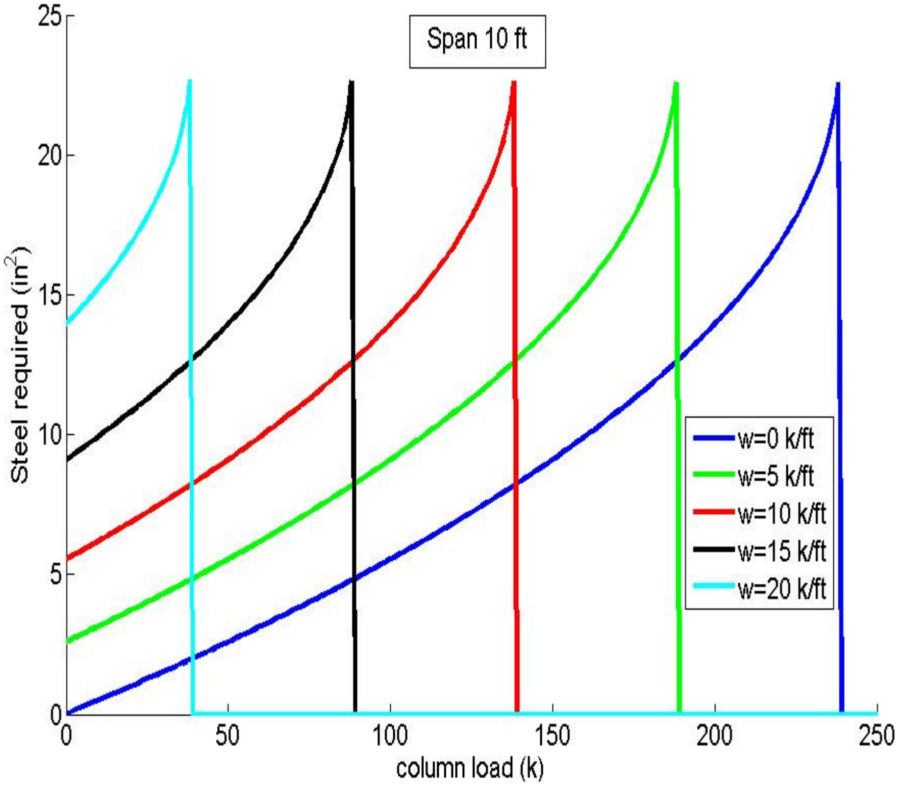

Figure A.1: Required steel with pinned/nominal conditions with a 10ft span

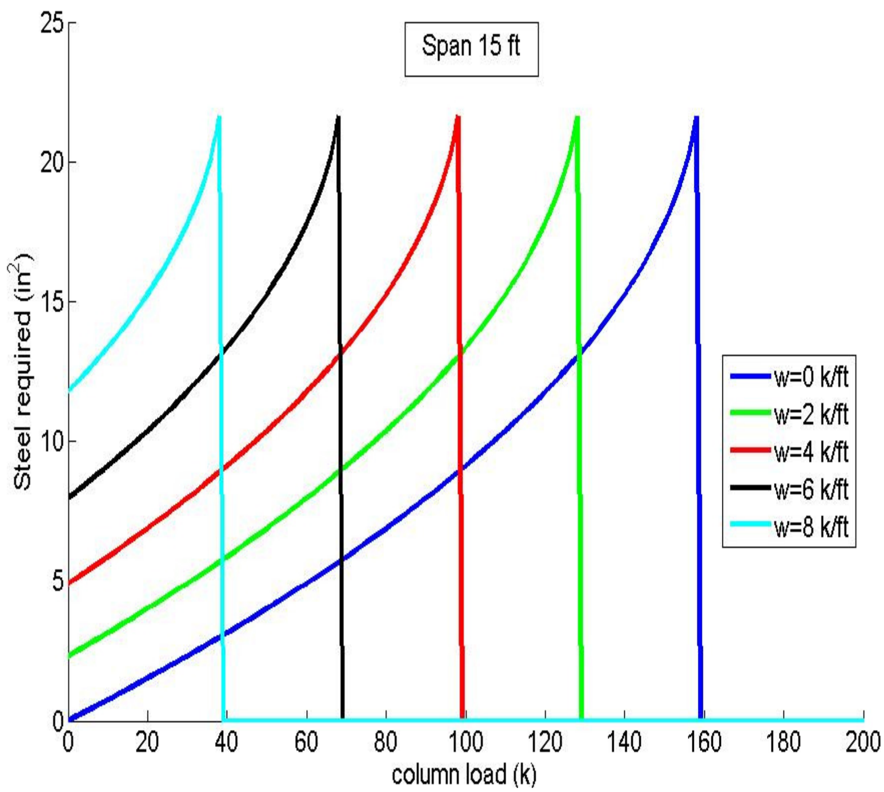

Figure A.2: Required steel with pinned/nominal conditions with a 15ft span

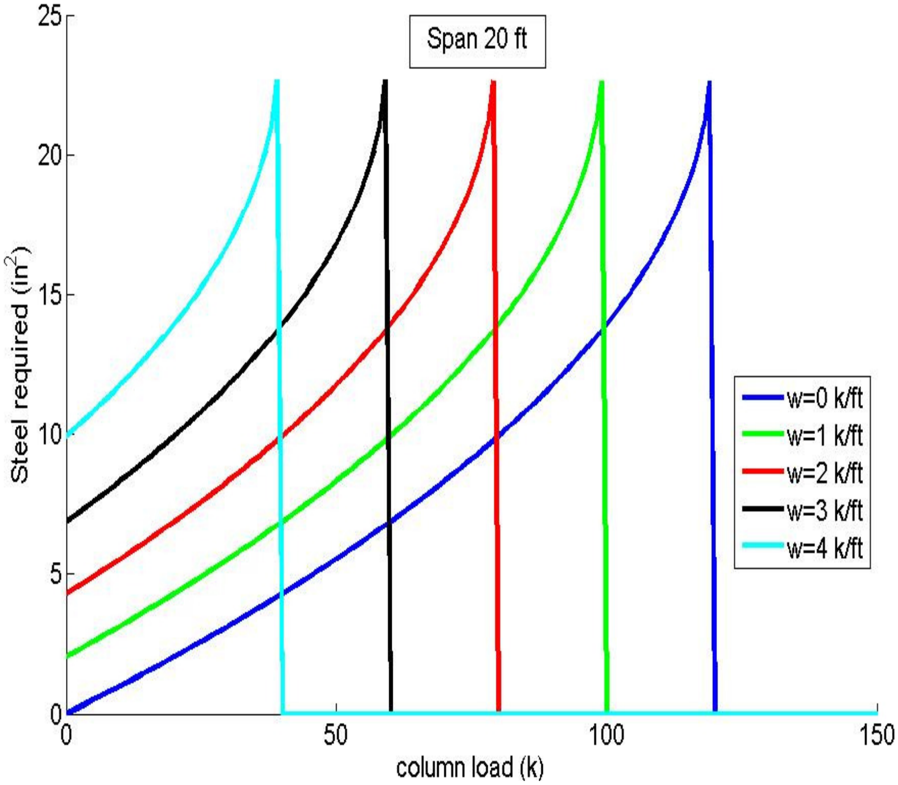

Figure A.3: Required steel with pinned/nominal conditions with a 20ft span

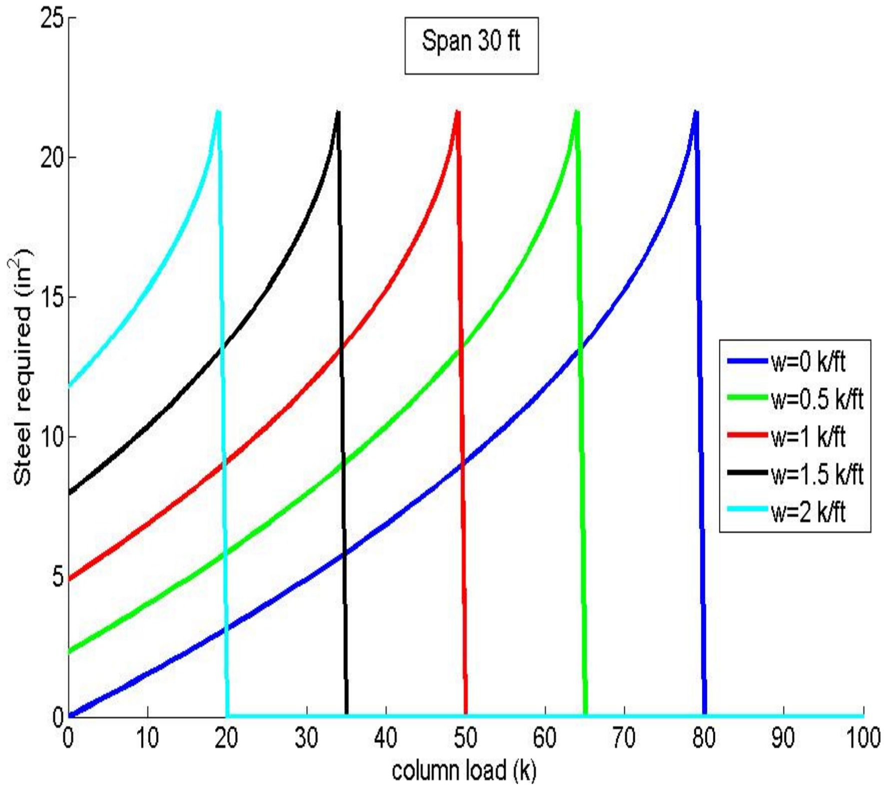

Figure A.4: Required steel with pinned/nominal conditions with a 30ft span

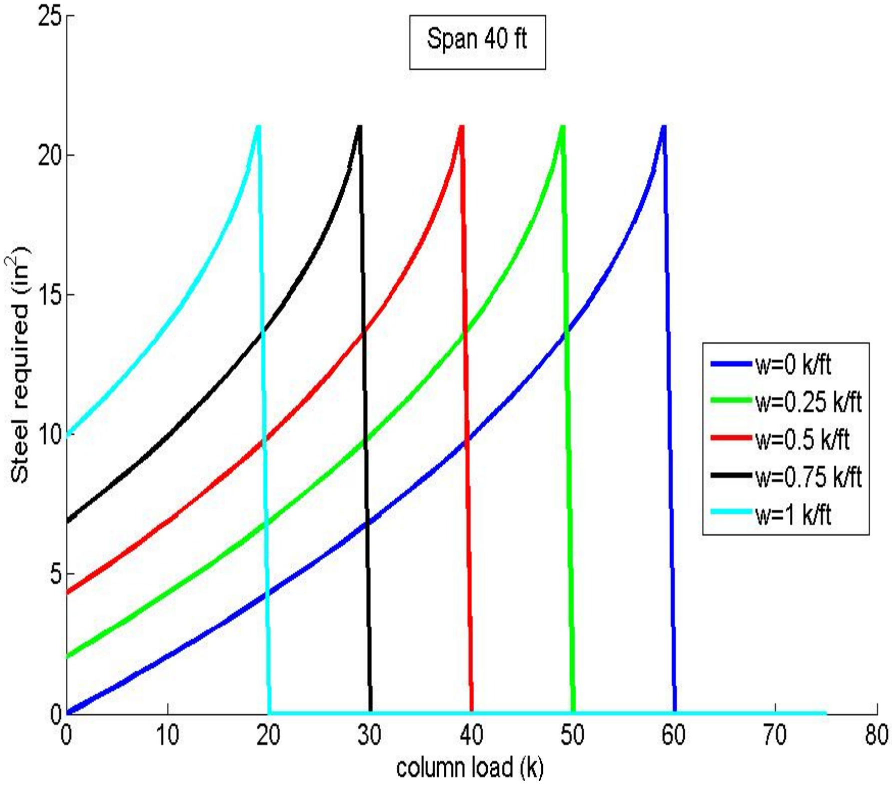

Figure A.5: Required steel with pinned/nominal conditions with a 40ft span

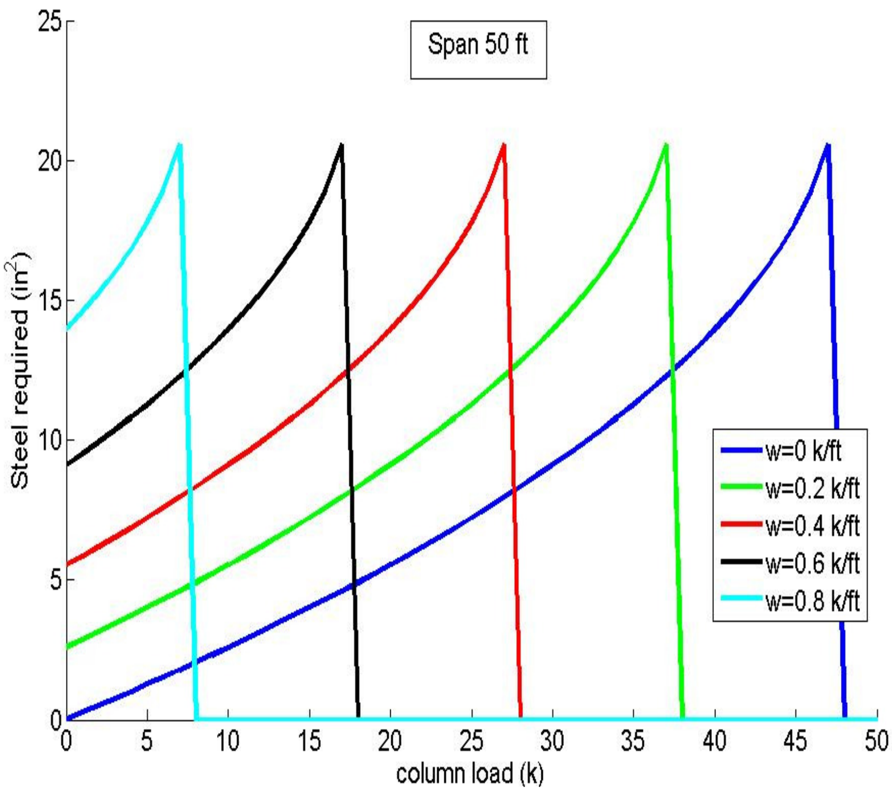

Figure A.6: Required steel with pinned/nominal conditions with a 50ft span

APPENDIX B

PINNED END / PLASTIC RESULTS

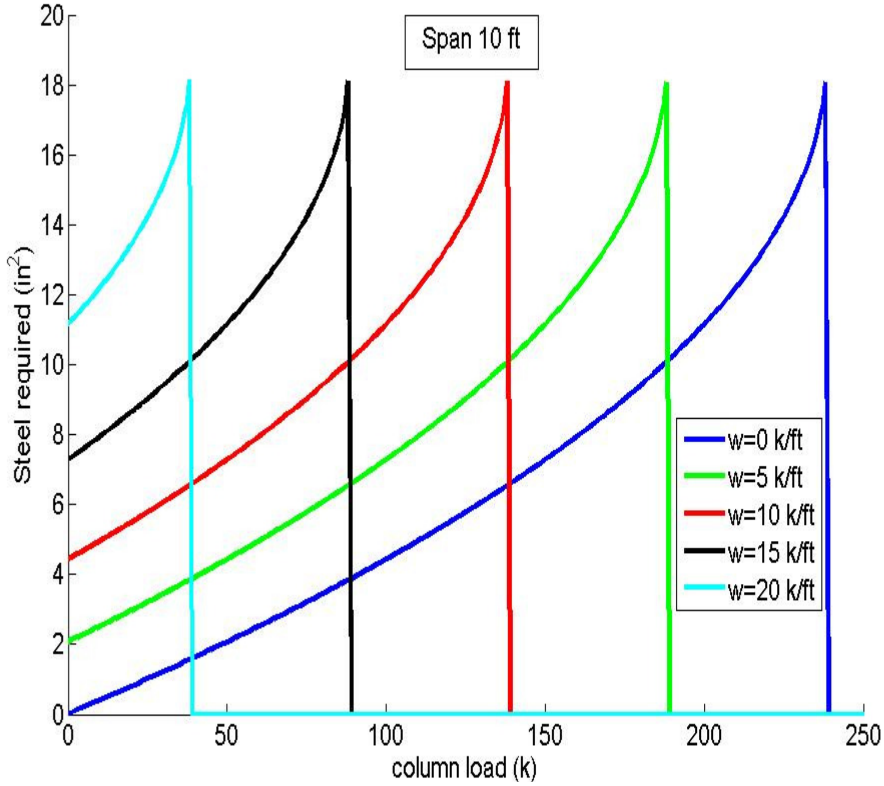

Figure B.1: Required steel with pinned/plastic conditions with a 10ft span

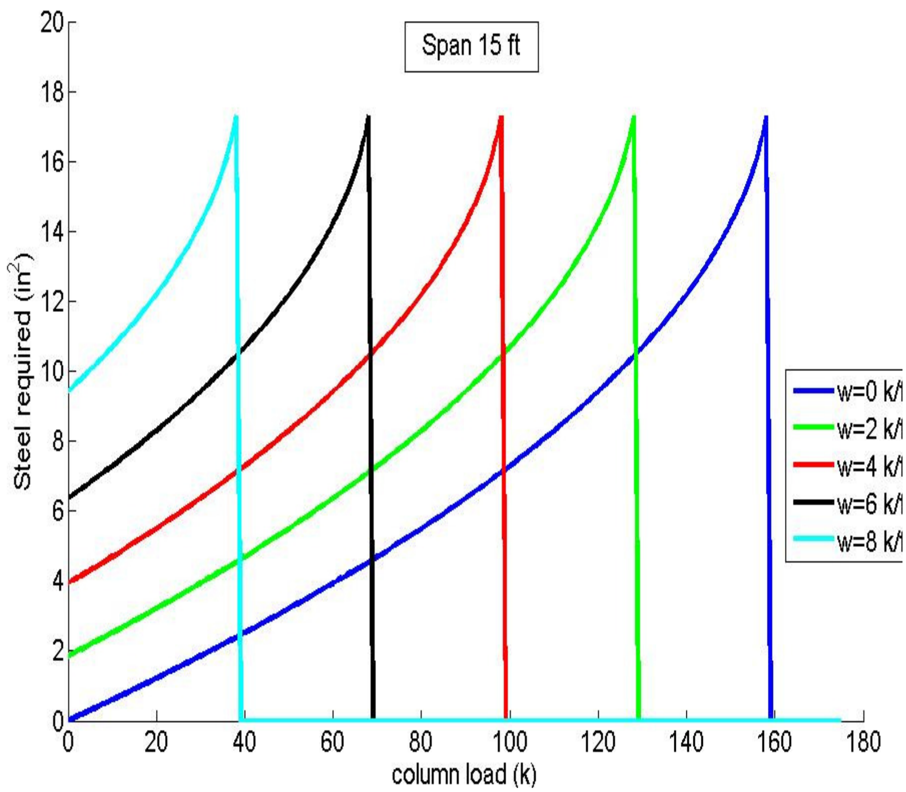

Figure B.2: Required steel with pinned/plastic conditions with a 15ft span

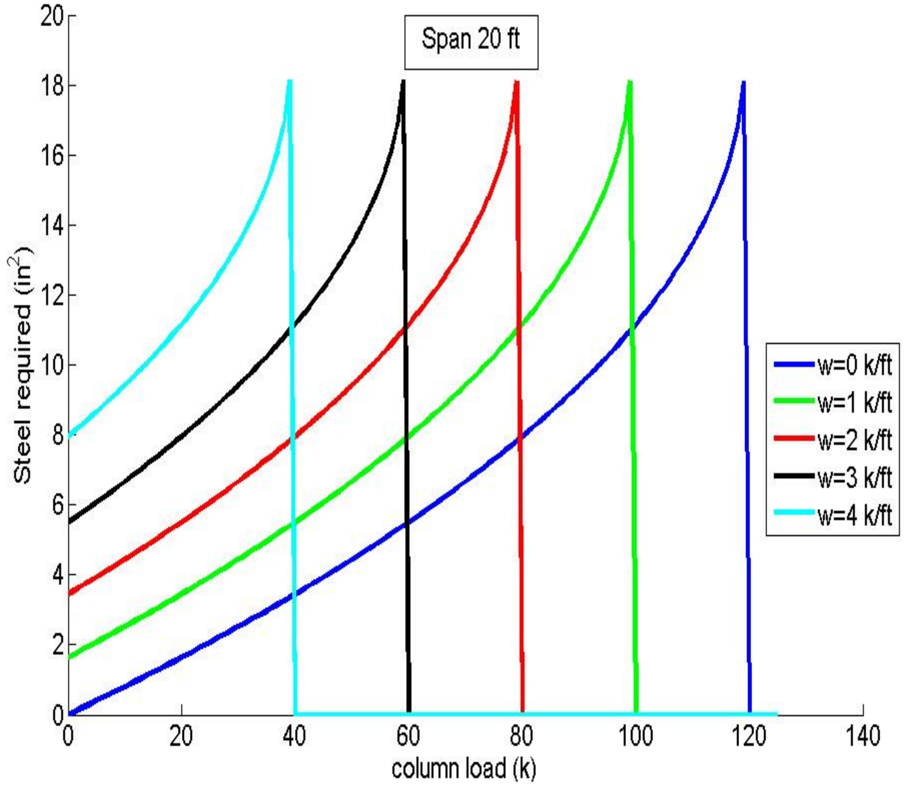

Figure B.3: Required steel with pinned/plastic conditions with a 20ft span

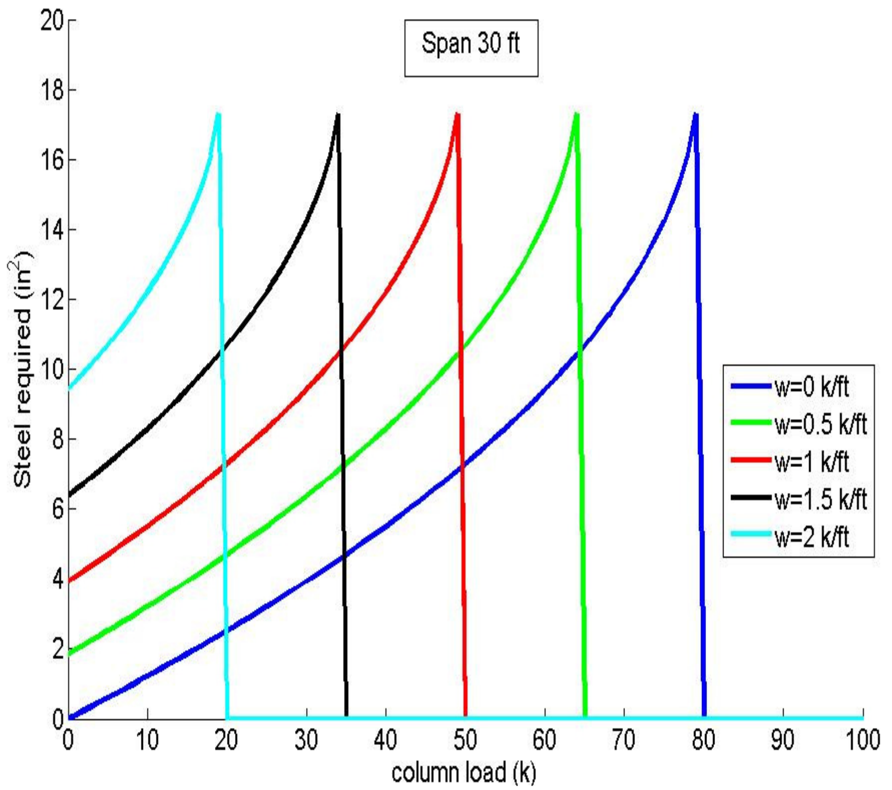

Figure B.4: Required steel with pinned/plastic conditions with a 30ft span

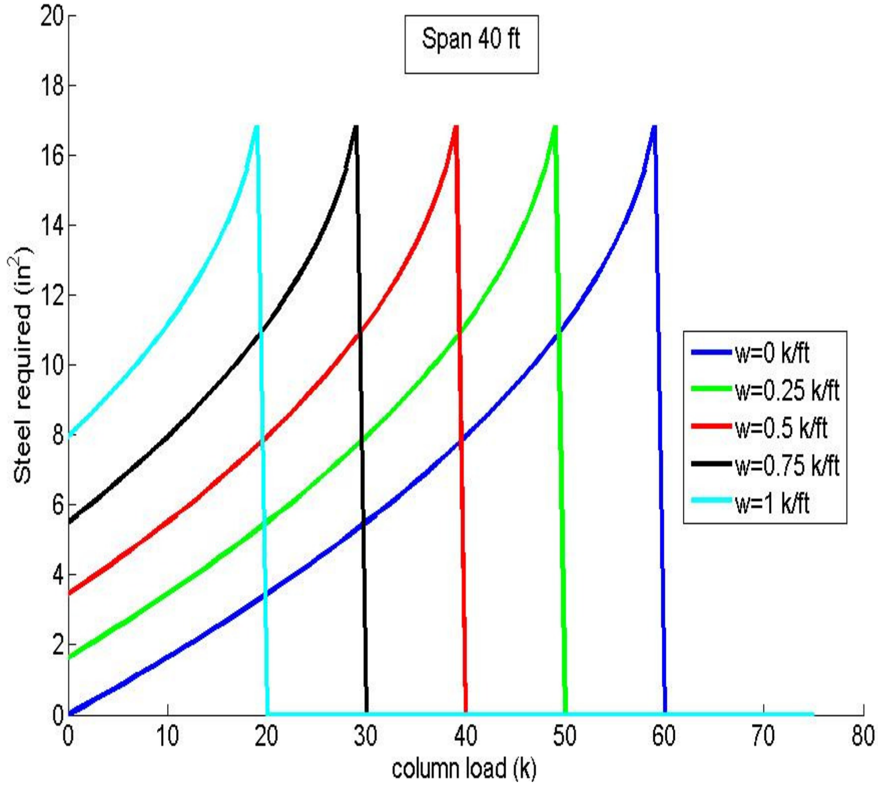

Figure B.5: Required steel with pinned/plastic conditions with a 40ft span

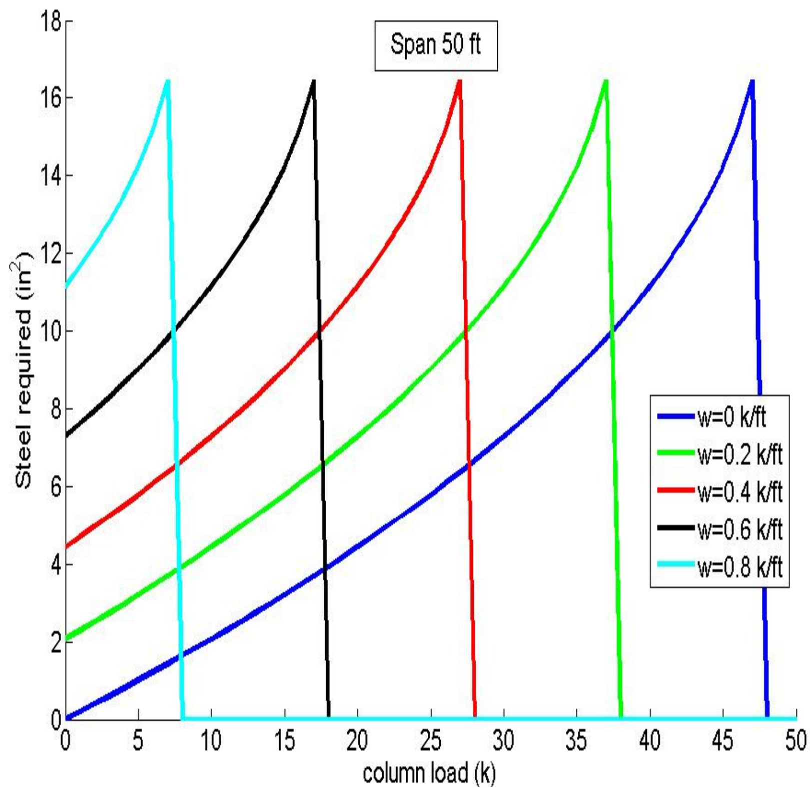

Figure B.6: Required steel with pinned/plastic conditions with a 50ft span

APPENDIX C

FIXED END / NOMINAL RESULTS

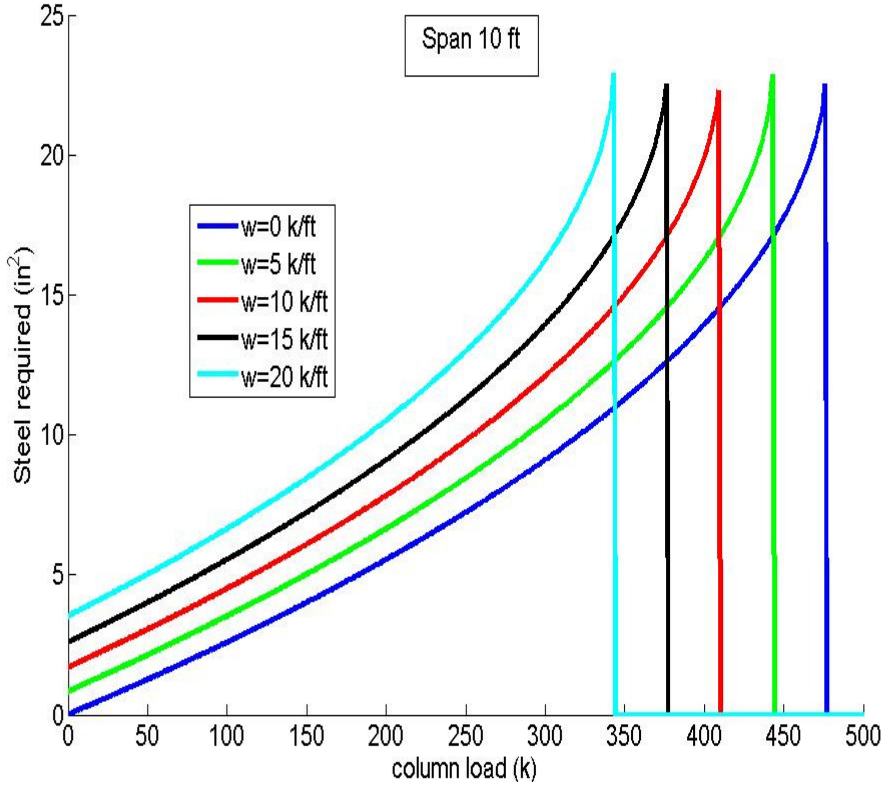

Figure C.1: Required steel with fixed/nominal conditions with a 10ft span

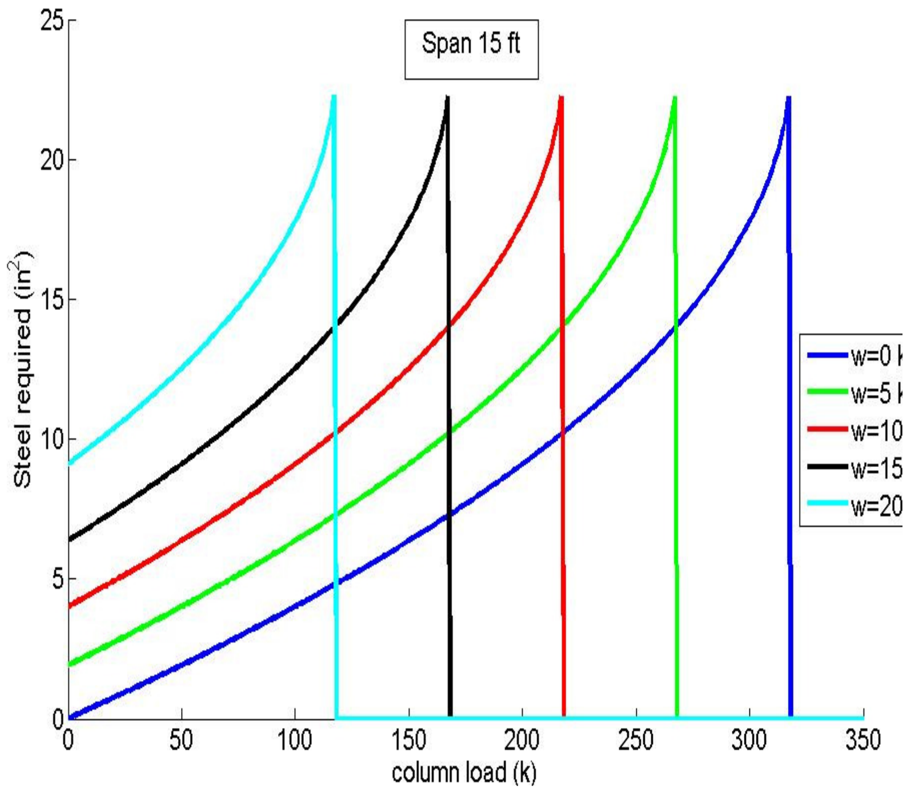

Figure C.2: Required steel with fixed/nominal conditions with a 15ft span

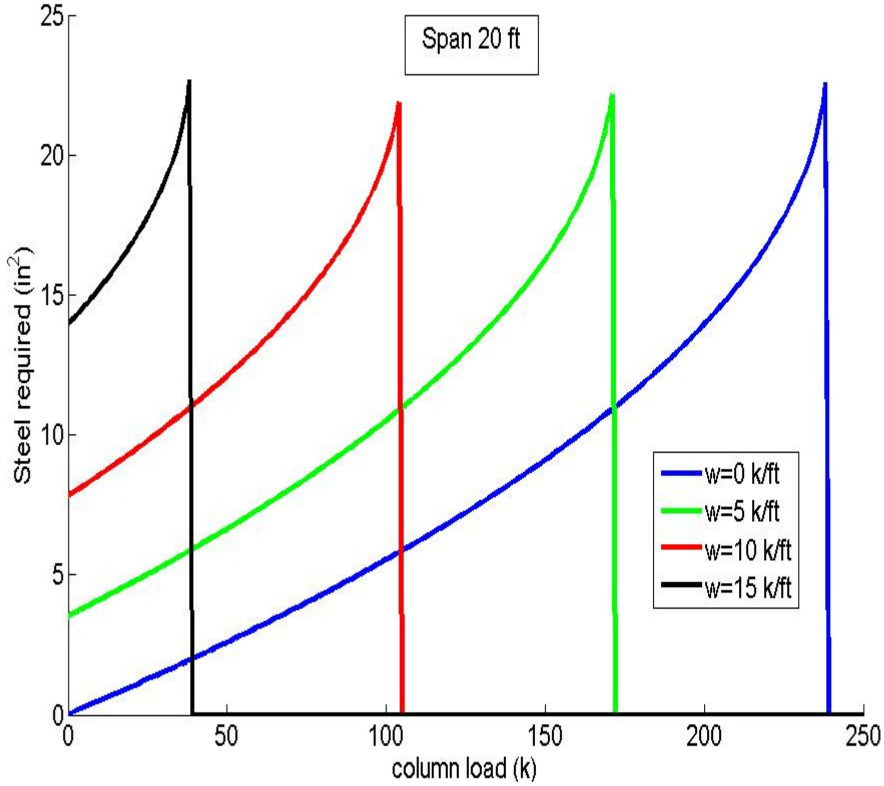

Figure C.3: Required steel with fixed/nominal conditions with a 20ft span

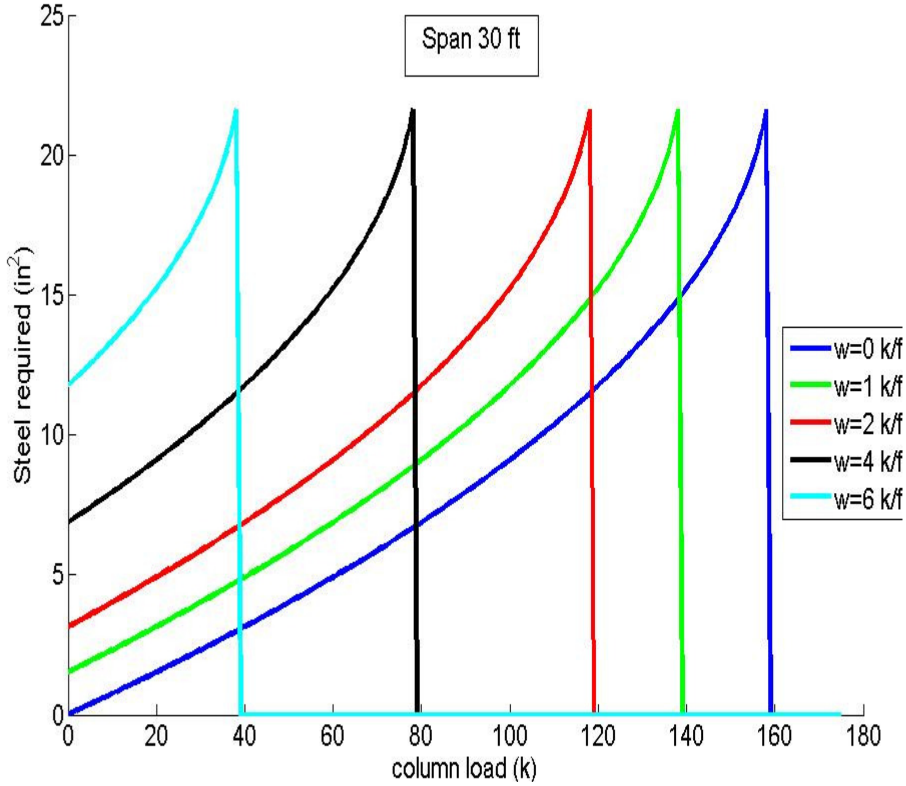

Figure C.4: Required steel with fixed/nominal conditions with a 30ft span

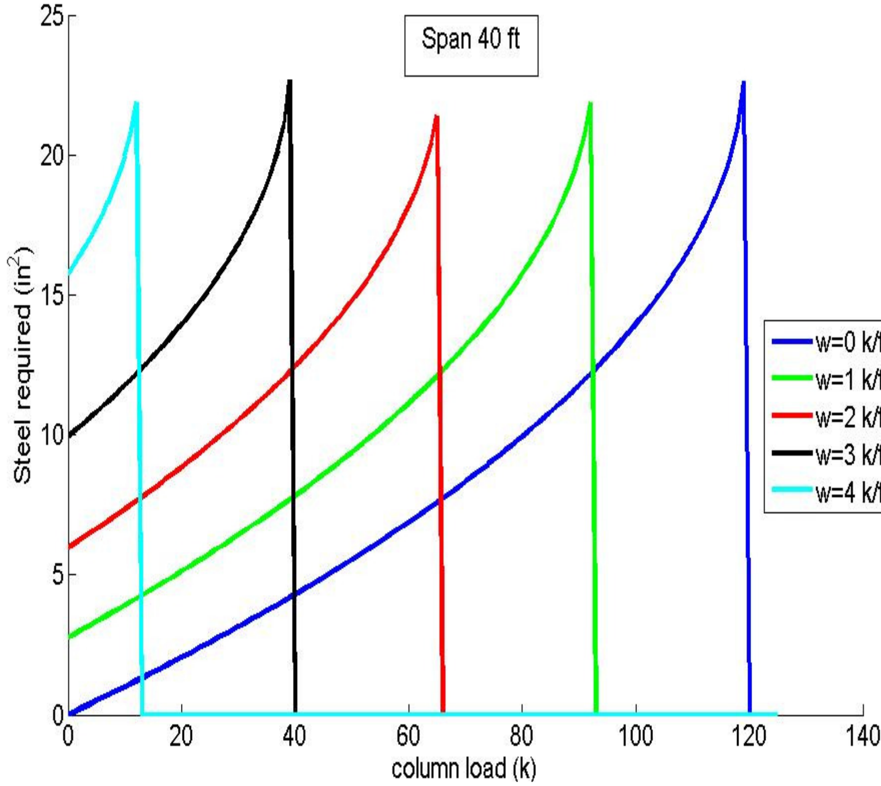

Figure C.5: Required steel with fixed/nominal conditions with a 40ft span

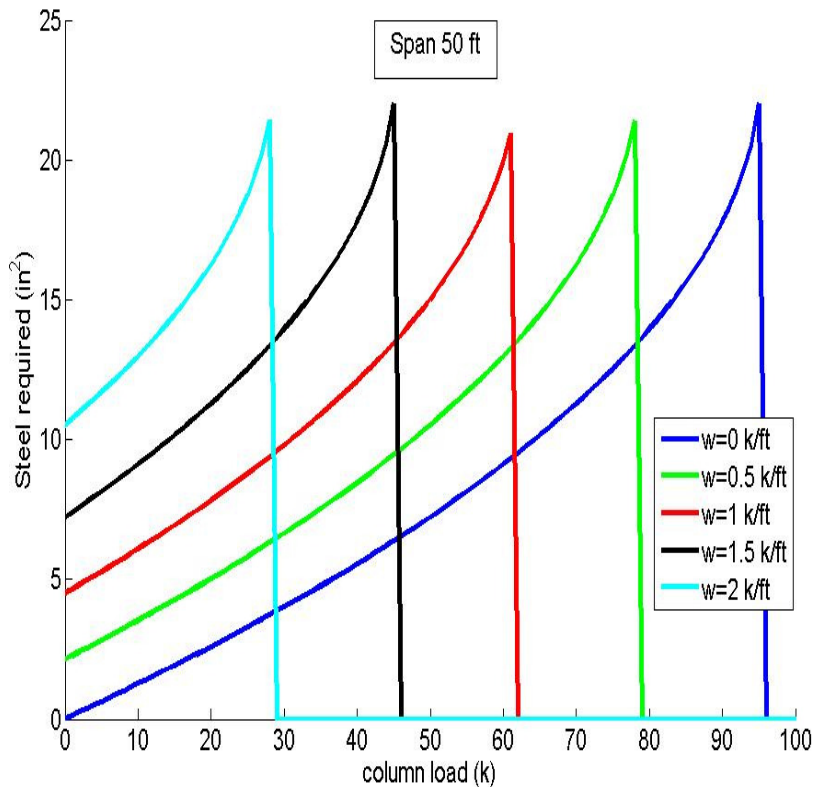

Figure C.6: Required steel with fixed/nominal conditions with a 50ft span

APPENDIX D

FIXED END / PLASTIC RESULTS

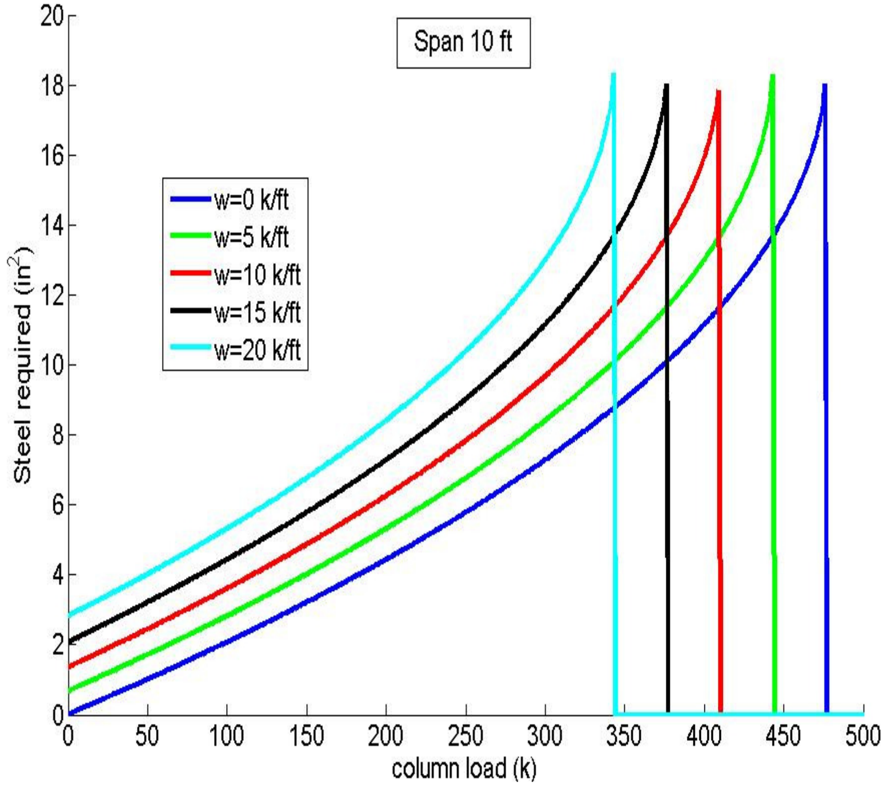

Figure D.1: Required steel with fixed/plastic conditions with a 10ft span

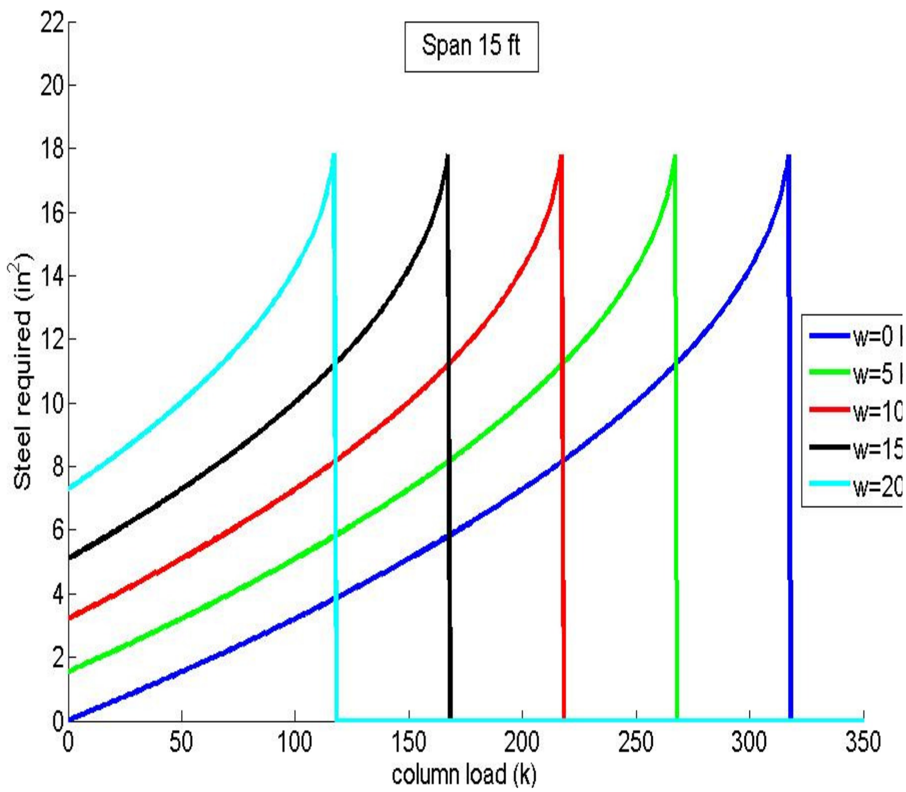

Figure D.2: Required steel with fixed/plastic conditions with a 15ft span

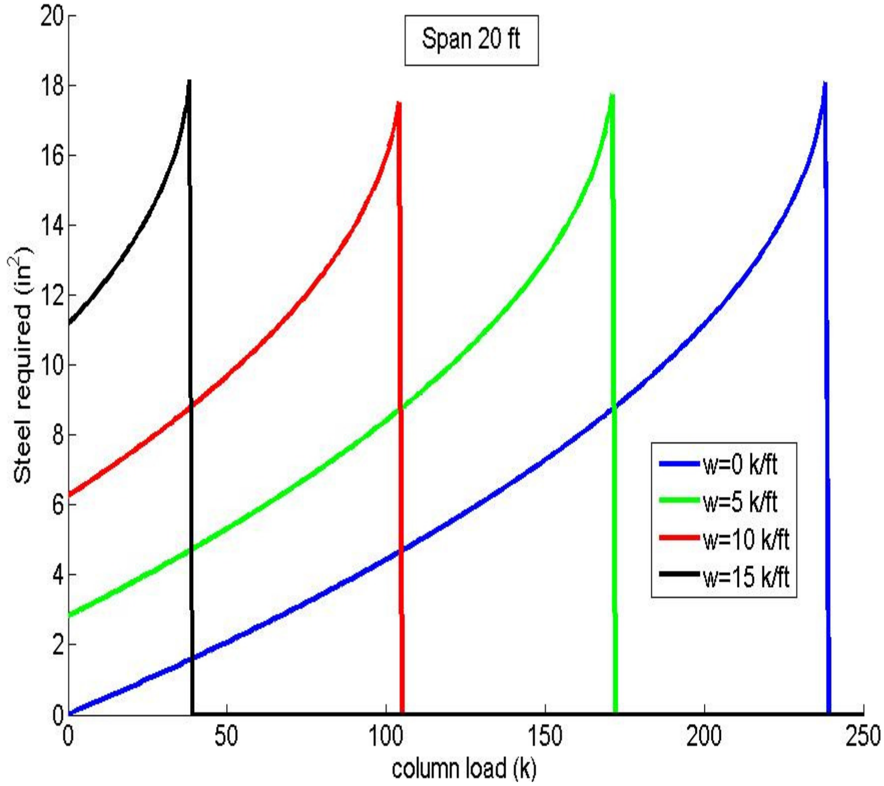

Figure D.3: Required steel with fixed/plastic conditions with a 20ft span

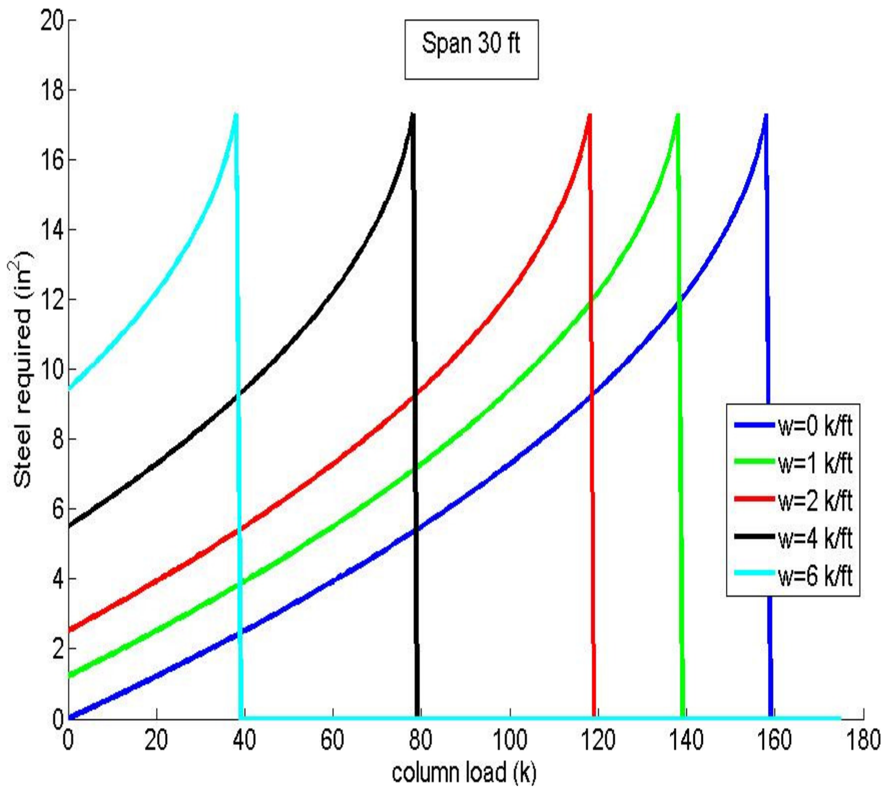

Figure D.4: Required steel with fixed/plastic conditions with a 30ft span

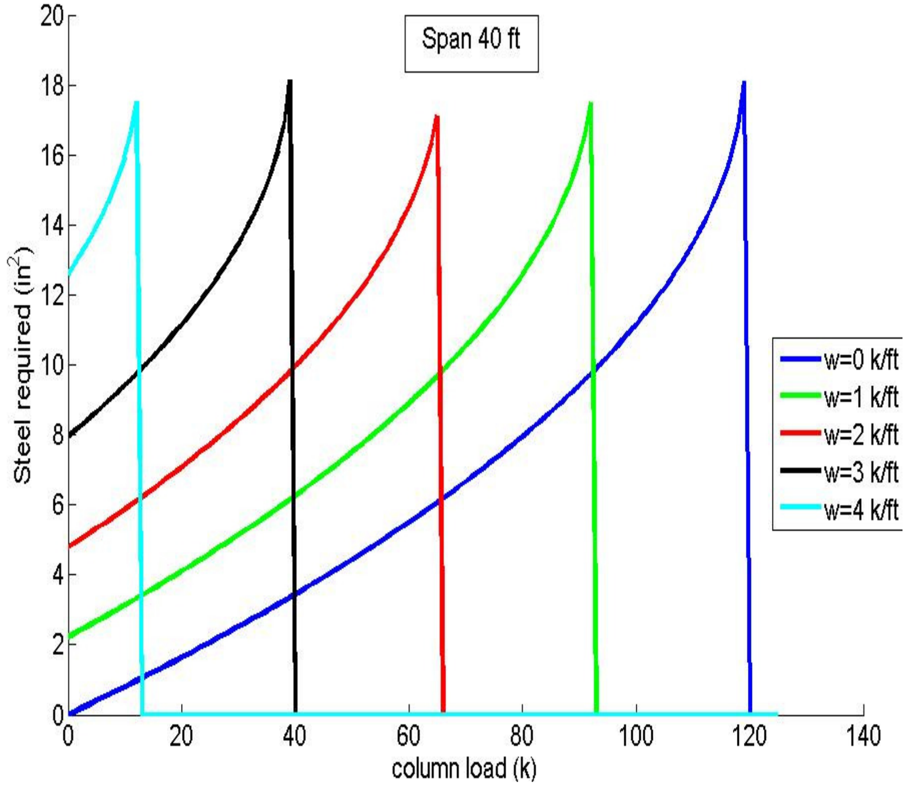

Figure D.5: Required steel with fixed/plastic conditions with a 40ft span

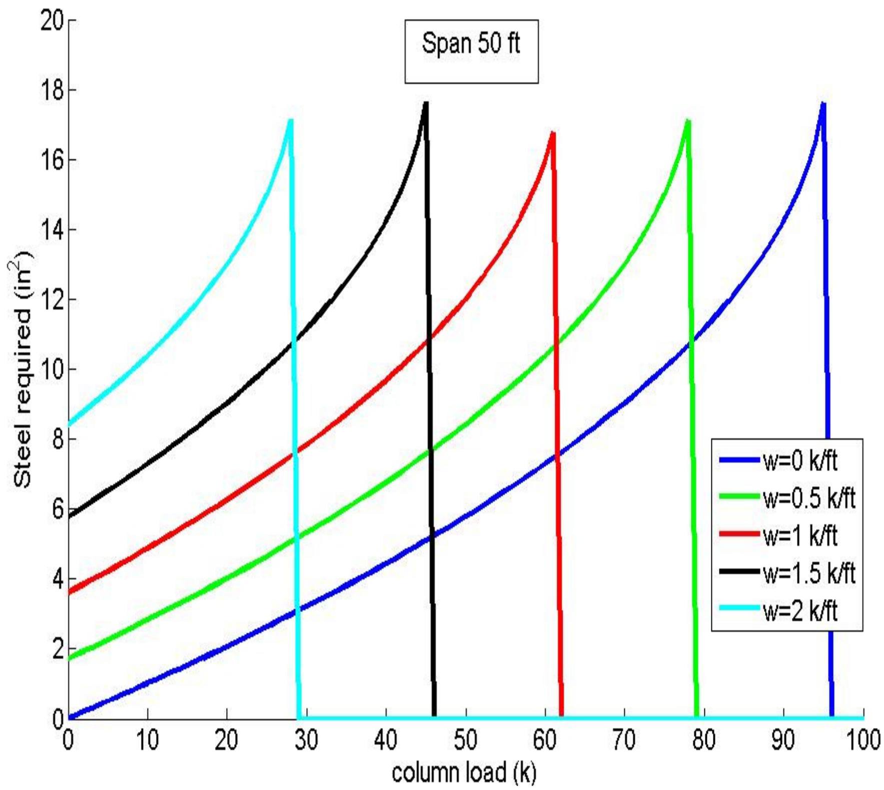

Figure D.6: Required steel with fixed/plastic conditions with a 50ft span

APPENDIX E

PINNED END / NOMINAL PROGRAM

```
function[]= pinyield(span,wx1,wx2,w,cx,c)
%function[]= beam(span,wx1,wx2,w,cx,c)
%This program was developed for the purpose of designing/selecting a
%beam of adequate strength for given conditions.
%All distances are measured from the far left end of the beam.
%Steel yielding strength assumed to be 50ksi.
%INPUT
%L: length of the beam
%wx1: vector of starting locations of uniform loads
%wx2: vector of ending locations of uniform loads
%w: vector of magnitudes of uniform loads
%cx: vector of locations of concentrated loads
%c: vector of magnitudes of concentrated loads
%OUTPUT
%Vu: maximum shear value in the beam
%Mu: maximum moment value in the beam
%Mn: actual moment capacity

Mu=zeros(1,length(c));
Mn=zeros(1,length(c));
AS=zeros(1,length(c));

for ii=1:1:length(c)
    L=2*span;
    z=1000*L;
    step=L/z;
    F=zeros(1,z);
    M=zeros(1,z);
    x=[0:step:L-step];
    V=zeros(1,z);

    for j=1:z:1
        F(j)=0;
        x(j)=0;
        V(j)=0;
    end

    for kk=1:1:length(w)
        q=round(wx1(kk)*z/L);
        if q==0
            q=1;
        end
        for k=q:1:round(wx2(kk)*z/L)
            F(k)=(wx2(kk)-wx1(kk)).*w(kk)./((wx2(kk)-wx1(kk))*z/L);
        end
    end

    F(round(cx*z/L))=c(ii);
    R1=(sum(w.*(wx2-wx1).*(L-wx2+((wx2-wx1)/2)))+sum(c(ii)*(L-cx)))/L;
    V(1)=0;
    V(2)=R1;
    M(1)=0;

    for i=3:1:z
        V(i)=V(i-1)-F(i-1);
        M(i)=(V(i)+V(i-1))*.5*step+M(i-1);
    end
```

```
    V(z)=0;
    M(z)=0;
    Mu(ii)=max(M);
    Vu(ii)=max(V);
    beam=zeros(1,z);
    %plot(x,V,'b',x,M,'r',x,beam,'k','LineWidth',3)
    %legend('Shear','Moment','Beam')

    fc1=4;
    h=24;
    b=20;
    As=0;
    fy=60;
    d=h-3.5;
    a=As*fy/(.85*b*fc1);
    Mn(ii)=As*fy*(d-a/2)/12;

    while Mn(ii)<Mu(ii)
        As=As+.01;
        a=As*fy/(.85*b*fc1);
        Mn(ii)=As*fy*(d-a/2)/12;
        if Mn(ii)<0
            As=0;
            break
        end
    end
AS(ii)=As;
end
hold on
plot(c,AS,'c')
xlabel('column load (k)')
ylabel('Steel required (in^2)')
legend('w=0 k/ft','w=0.2 k/ft','w=0.4 k/ft','w=0.6 k/ft','w=0.8 k/ft','w=10
k/ft')

as(1)=AS(1)-(AS(2)-AS(1));
j=2;
for iii=10:10:length(c)
    if AS(iii)>0
        as(j)=AS(iii);
        j=j+1;
    end
end
as

jj=2;
while AS(jj)>0
    Pmax=c(jj);
    jj=jj+1;
end
Pmax
```

APPENDIX F

PINNED END / PLASTIC PROGRAM

```
function[]= pinplastic(span,wx1,wx2,w,cx,c)
%function[]= beam(span,wx1,wx2,w,cx,c)
%This program was developed for the purpose of designing/selecting a
%beam of adequate strength for given conditions.
%All distances are measured from the far left end of the beam.
%Steel yielding strength assumed to be 50ksi.
%INPUT
%L: length of the beam
%wx1: vector of starting locations of uniform loads
%wx2: vector of ending locations of uniform loads
%w: vector of magnitudes of uniform loads
%cx: vector of locations of concentrated loads
%c: vector of magnitudes of concentrated loads
%OUTPUT
%Vu: maximum shear value in the beam
%Mu: maximum moment value in the beam
%Mn: actual moment capacity

Mu=zeros(1,length(c));
Mn=zeros(1,length(c));
AS=zeros(1,length(c));

for ii=1:1:length(c)
    L=2*span;
    z=1000*L;
    step=L/z;
    F=zeros(1,z);
    M=zeros(1,z);
    x=[0:step:L-step];
    V=zeros(1,z);

    for j=1:z:1
        F(j)=0;
        x(j)=0;
        V(j)=0;
    end

    for kk=1:1:length(w)
        q=round(wx1(kk)*z/L);
        if q==0
            q=1;
        end
        for k=q:1:round(wx2(kk)*z/L)
            F(k)=(wx2(kk)-wx1(kk)).*w(kk)./((wx2(kk)-wx1(kk))*z/L);
        end
    end

    F(round(cx*z/L))=c(ii);
    R1=(sum(w.*(wx2-wx1).*(L-wx2+((wx2-wx1)/2)))+sum(c(ii)*(L-cx)))/L;
    V(1)=0;
    V(2)=R1;
    M(1)=0;

    for i=3:1:z
        V(i)=V(i-1)-F(i-1);
        M(i)=(V(i)+V(i-1))*.5*step+M(i-1);
    end
```

```
    V(z)=0;
    M(z)=0;
    Mu(ii)=max(M);
    Vu(ii)=max(V);
    beam=zeros(1,z);
    %plot(x,V,'b',x,M,'r',x,beam,'k','LineWidth',3)
    %legend('Shear','Moment','Beam')

    fc1=4;
    h=24;
    b=20;
    As=0;
    fy=60;
    fy=1.25*fy;
    d=h-3.5;
    a=As*fy/(.85*b*fc1);
    Mn(ii)=As*fy*(d-a/2)/12;

    while Mn(ii)<Mu(ii)
        As=As+.01;
        a=As*fy/(.85*b*fc1);
        Mn(ii)=As*fy*(d-a/2)/12;
        if Mn(ii)<0
            As=0;
            break
        end
    end
AS(ii)=As;
end
hold on
plot(c,AS,'c')
xlabel('column load (k)')
ylabel('Steel required (in^2)')
legend('w=0 k/ft','w=0.2 k/ft','w=0.4 k/ft','w=0.6 k/ft','w=0.8 k/ft','w=10
k/ft')

as(1)=AS(1)-(AS(2)-AS(1));
j=2;
for iii=10:10:length(c)
    if AS(iii)>0
        as(j)=AS(iii);
        j=j+1;
    end
end
as

jj=2;
while AS(jj)>0
    Pmax=c(jj);
    jj=jj+1;
end
Pmax
```

APPENDIX G

FIXED END / NOMINAL PROGRAM

```
function[]= fixyield(span,wx1,wx2,w,cx,c)
%function[]= beam(span,wx1,wx2,w,cx,c)
%This program was developed for the purpose of designing/selecting a
%beam of adequate strength for given conditions.
%All distances are measured from the far left end of the beam.
%Steel yielding strength assumed to be 50ksi.
%INPUT
%L: length of the beam
%wx1: vector of starting locations of uniform loads
%wx2: vector of ending locations of uniform loads
%w: vector of magnitudes of uniform loads
%cx: vector of locations of concentrated loads
%c: vector of magnitudes of concentrated loads
%OUTPUT
%Vu: maximum shear value in the beam
%Mu: maximum moment value in the beam
%Mn: actual moment capacity

Mu=zeros(1,length(c));
Mn=zeros(1,length(c));
AS=zeros(1,length(c));

for ii=1:1:length(c)
    L=2*span;
    z=1000*L;
    step=L/z;
    F=zeros(1,z);
    M=zeros(1,z);
    x=[0:step:L-step];
    V=zeros(1,z);

    for j=1:z:1
        F(j)=0;
        x(j)=0;
        V(j)=0;
    end

    for kk=1:1:length(w)
        q=round(wx1(kk)*z/L);
        if q==0
            q=1;
        end
        for k=q:1:round(wx2(kk)*z/L)
            F(k)=(wx2(kk)-wx1(kk)).*w(kk)./((wx2(kk)-wx1(kk))*z/L);
        end
    end

    F(round(cx*z/L))=c(ii);
    R1=(sum(w.*(wx2-wx1).*(L-wx2+((wx2-wx1)/2)))+sum(c(ii)*(L-cx)))/L;
    V(1)=0;
    V(2)=R1;
    M(1)=0;

    for i=3:1:z
        V(i)=V(i-1)-F(i-1);
        M(i)=(V(i)+V(i-1))*.5*step+M(i-1);
    end
```

```
    V(z)=0;
    M(z)=0;
    M=M-c(ii)*span/4-w*span*span/3;
    M(1)=0;
    M(z)=0;
    Mu(ii)=max(M);
    Vu(ii)=max(V);
    beam=zeros(1,z);
    %plot(x,V,'b',x,M,'r',x,beam,'k','LineWidth',3)
    %legend('Shear','Moment','Beam')

    fc1=4;
    h=24;
    b=20;
    As=0;
    fy=60;
    d=h-3.5;
    a=As*fy/(.85*b*fc1);
    Mn(ii)=As*fy*(d-a/2)/12;

    while Mn(ii)<Mu(ii)
        As=As+.01;
        a=As*fy/(.85*b*fc1);
        Mn(ii)=As*fy*(d-a/2)/12;
        if Mn(ii)<0
            As=0;
            break
        end
    end
AS(ii)=As;
end
hold on
plot(c,AS,'c')
xlabel('column load (k)')
ylabel('Steel required (in^2)')
legend('w=0 k/ft','w=0.5 k/ft','w=1 k/ft','w=1.5 k/ft','w=2 k/ft','w=10
k/ft')

as(1)=AS(1)-(AS(2)-AS(1));
j=2;
for iii=10:10:length(c)
    if AS(iii)>0
        as(j)=AS(iii);
        j=j+1;
    end
end
as
jj=2;
while AS(jj)>0
    Pmax=c(jj);
    jj=jj+1;
end
Pmax
```

APPENDIX H

FIXED END / PLASTIC PROGRAM

```
function[]= fixplastic(span,wx1,wx2,w,cx,c)
%function[]= beam(span,wx1,wx2,w,cx,c)
%This program was developed for the purpose of designing/selecting a
%beam of adequate strength for given conditions.
%All distances are measured from the far left end of the beam.
%Steel yielding strength assumed to be 50ksi.
%INPUT
%L: length of the beam
%wx1: vector of starting locations of uniform loads
%wx2: vector of ending locations of uniform loads
%w: vector of magnitudes of uniform loads
%cx: vector of locations of concentrated loads
%c: vector of magnitudes of concentrated loads
%OUTPUT
%Vu: maximum shear value in the beam
%Mu: maximum moment value in the beam
%Mn: actual moment capacity

Mu=zeros(1,length(c));
Mn=zeros(1,length(c));
AS=zeros(1,length(c));

for ii=1:1:length(c)
    L=2*span;
    z=1000*L;
    step=L/z;
    F=zeros(1,z);
    M=zeros(1,z);
    x=[0:step:L-step];
    V=zeros(1,z);

    for j=1:z:1
        F(j)=0;
        x(j)=0;
        V(j)=0;
    end

    for kk=1:1:length(w)
        q=round(wx1(kk)*z/L);
        if q==0
            q=1;
        end
        for k=q:1:round(wx2(kk)*z/L)
            F(k)=(wx2(kk)-wx1(kk)).*w(kk)./((wx2(kk)-wx1(kk))*z/L);
        end
    end

    F(round(cx*z/L))=c(ii);
    R1=(sum(w.*(wx2-wx1).*(L-wx2+((wx2-wx1)/2)))+sum(c(ii)*(L-cx)))/L;
    V(1)=0;
    V(2)=R1;
    M(1)=0;

    for i=3:1:z
        V(i)=V(i-1)-F(i-1);
        M(i)=(V(i)+V(i-1))*.5*step+M(i-1);
    end
```

```
    V(z)=0;
    M(z)=0;
    M=M-c(ii)*span/4-w*span*span/3;
    M(1)=0;
    M(z)=0;
    Mu(ii)=max(M);
    Vu(ii)=max(V);
    beam=zeros(1,z);
    %plot(x,V,'b',x,M,'r',x,beam,'k','LineWidth',3)
    %legend('Shear','Moment','Beam')

    fc1=4;
    h=24;
    b=20;
    As=0;
    fy=60;
    fy=1.25*fy;
    d=h-3.5;
    a=As*fy/(.85*b*fc1);
    Mn(ii)=As*fy*(d-a/2)/12;

    while Mn(ii)<Mu(ii)
        As=As+.01;
        a=As*fy/(.85*b*fc1);
        Mn(ii)=As*fy*(d-a/2)/12;
        if Mn(ii)<0
            As=0;
            break
        end
    end
AS(ii)=As;
end
hold on
plot(c,AS,'c')
xlabel('column load (k)')
ylabel('Steel required (in^2)')
legend('w=0 k/ft','w=0.5 k/ft','w=1 k/ft','w=1.5 k/ft','w=2 k/ft','w=10
k/ft')

as(1)=AS(1)-(AS(2)-AS(1));
j=2;
for iii=10:10:length(c)
    if AS(iii)>0
        as(j)=AS(iii);
        j=j+1;
    end
end
as
jj=2;
while AS(jj)>0
    Pmax=c(jj);
    jj=jj+1;
end
Pmax
```